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# THE 1985 HIGHWAY CAPACITY MANUAL

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## -- A SUMMARY --

APRIL 1986

U.S. Department of Transportation
Federal Highway Administration

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# THE 1985 HIGHWAY CAPACITY MANUAL

## A SUMMARY

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APRIL 1986

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Note: Figures and Tables with hyphenated numbers
are from Highway Capacity Manual, Special Report 209,
Transportation Research Board, Washington, D.C. (1985).

# THE 1985 HIGHWAY CAPACITY MANUAL

## --A Summary--

## FOREWORD

The third edition of the Highway Capacity Manual(HCM) was published in August, 1985, after more than 15 years of study by a wide variety of research and government agencies. Published under the guidance of the Transportation Research Board's Committee on Highway Capacity and Quality of Service, the 1985 HCM reflects a level of sophistication and understanding that will allow more precise and accurate analysis of highway facilities and operating conditions thereon. Some procedures have become computationally complex, and the utility of microcomputer software developed to replicate the procedures of the 1985 HCM will be of great importance to users of the new manual.

This document represents a summary of the 1985 HCM. It has been prepared especially for personnel of the Federal Highway Administration to assist the transition process from the 1965 HCM to the 1985 HCM. This summary also highlights the major differences between the 1965 HCM and the 1985 HCM, as well as, the key features and principal contents of the 1985 HCM.

## DEVELOPMENT OF HIGHWAY CAPACITY SINCE 1950

The first edition of the Highway Capacity Manual was published in 1950 by the then-Bureau of Public Roads. At the time, there were no nationally-accepted standards for design, and consequently, very little uniformity in the way highways were designed and constructed. The 1950 HCM was primarily a design document, and did not address all types of traffic facilities completely.

The basic concept of highway capacity was introduced and defined. Actually, the 1950 HCM used three different kinds of capacity:

1. Practical Capacity
2. Possible Capacity
3. Ideal Capacity

"Practical capacity" was a forerunner of the Level of Service (LOS) concepts and was the capacity which could be achieved while maintaining "reasonable" operating quality. "Possible capacity" was analogous to the present use of "capacity", while "ideal capacity" referred to a condition with ideal geometric and traffic characteristics.

Work on the second edition of the Highway Capacity Manual began in 1953, and culminated in the publication of the 1965 HCM. When it appeared, the second edition of the HCM was a dramatic leap forward in the state-of-the-art of highway design and analysis. The most significant improvements included:

1. The use of the HCM as an operational analysis tool was expanded.

2. The 1965 HCM reflected the growing importance of limited access facilities, and devoted three chapters to freeways and their components.

3. The concept of Level of Service was introduced as a means to "grade" the operational quality of service provided by a facility to drivers and passengers.

4. The data upon which procedures were based was far superior in quantity and quality to that available for the 1950 HCM.

The Level of Service concept was most important. It broadened the use of procedures in operational analysis, and created a quality scale which has become an integral part of the traffic engineering profession. It has become the major descriptor by which traffic engineers communicate with decision-makers and public groups.

Just as work on the second edition of the HCM began shortly after the publication of the first, work on the third edition started only a few years after the publication of the 1965 HCM. In early 1970's, the National Cooperative Highway Research Program and the Federal Highway Administration began to let major contracts to study various aspects of highway capacity analysis. Table 1 contains just a partial list of the subjects that were studied in the period between 1970 and 1984, all of which provided vital inputs to the 1985 HCM.

In 1980, the Transportation Research Board issued Circular 212, which contained a series of "interim materials" for highway capacity analysis. These essentially represented a partial compilation of the research up to the late 1970's. It gave practicing professionals early access to new material, and it gave the members of the Highway Capacity and Quality of Service Committee a chance to receive feedback on procedures it was considering.

**TABLE 1**

**RESEARCH LEADING TO THE 1985 HCM**

| SPONSOR | RESEARCH AGENCY | TOPIC |
|---|---|---|
| NCHRP | TTRC, Polytechnic | Weaving Area Operations Study (1971-74) |
| NCHRP<br>FHWA | Midwest Research Institute | Simulation of Multilane, Mixed-Vehicle Traffic Streams (1973-76) |
| NCHRP | Penn State | Truck Weight-Horsepower Relationships (1975-77) |
| NCHRP | Wilbur Smith & Assoc. | Bus Operations on Highways (1974-77) |
| FHWA | TTRC, Polytechnic | Freeway Capacity Analysis Procedures (1977-79) |
| NCHRP | JHK & Assoc. | Towards a New Highway Capacity Manual (1976-79) |
| FHWA | PRC/Voorhees | Arterials (1977-82) |
| NCHRP | JHK & Assoc. | Calibration of a Critical Movement Analysis for the New HCM (1980-82) |
| FHWA | J. Leisch & Assoc. | Development of a New Weaving Area Capacity Technique (1980-1983) |
| NCHRP | TTI, Texas A & M | Capacity of Rural Two-Lane, Two-Way Highways (1981-83) |
| FHWA | TTI, Texas A & M<br>Herbert G. Whyte<br>PRC/Voorhees<br>Institute for Research | Calibration of Passenger Car Equivalents (1981-82) |
| FHWA | JHK & Assoc. | Comparison of Weaving Area Analysis Procedures (1983-85) |
| NCHRP | TTRC, Polytechnic | Final Preparation of a New HCM (1980-85) |

The third edition of the Highway Capacity Manual marks a significant departure from previous editions, both in form and content. Several new chapters have been added, and others have been significantly expanded. Some procedures have been radically changed from previous editions, while others are similar in form to methods in the 1965 HCM. Key features of the new manual, are summarized below:

1. The 1985 HCM is published in loose-leaf form. As new material or research becomes available, individual chapters will be revised, without waiting for a revision of the entire manual.

2. The 1985 HCM adds significant material in areas not covered by previous editions, including Unsignalized Intersections, Arterials, Transit, and Pedestrians.

3. Critical Movement Analysis has been adopted and calibrated as the basis for Signalized Intersection Analysis. These techniques provide a more precise analytic model for intersection operations, and are sensitive to a wide range of geometric, traffic, and control variables which influence intersection performance.

4. The Level of Service concept is continued, and new parameters are introduced to better quantify service to the facility user.

5. Worksheets are provided for many computational procedures, as are step-by-step instructions for performing calculations.

6. Separate sections of each chapter cover basic principles and operating characteristics, a textbook presentation of methodology, step-by-step computational instructions, and sample calculations.

7. Procedures represent more substantial data bases than have ever been available in the past, although there are still many areas in which additional data would have been useful.

The 1985 HCM is another benchmark in the evolution of capacity analysis procedures and their application. The manual represents the best available knowledge and judgment of the many individuals and organizations which contributed to its productions.



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## CHAPTER 1: INTRODUCTION, DEFINITIONS AND CONCEPTS, APPLICATIONS

The first chapter of the HCM defines basic concepts of capacity analysis, and gives a generic overview of traffic characteristics. It also gives a discussion of the contents of the HCM, and a brief treatment of how the manual should be used and interpreted.

### INTRODUCTION

The fourteen chapters of the HCM represent revisions of material contained in the earlier editions, and new material reflecting the many changes in the characteristics of travel and in the information needed to conduct highway capacity analyses. They are organized according to facility type as shown in Table 1-1.

**TABLE 1-1. TYPES OF FACILITIES***

| FACILITY | CHAPTER |
|---|---|
| *Uninterrupted Flow Facilities* | |
| Freeways | |
| Basic freeway segments | 3 |
| Weaving areas | 4 |
| Ramps and ramp junctions | 5 |
| Freeway systems | 6 |
| Multilane Highways | 7 |
| Two-Lane Highways | 8 |
| *Interrupted Flow Facilities* | |
| Signalized Intersections | 9 |
| Unsignalized Intersections (2-way STOP-YIELD-controlled approaches; 4-way STOP-controlled intersections) | 10 |
| Arterials | 11 |
| Transit | 12 |
| Pedestrians | 13 |
| Bicycles | 14 |

Each of the procedural chapters in generally organized into four distinct parts:

1. Introduction - This section describes the basic characteristics, concepts, and philosophies of capacity analysis as applied to the subject type of facility.

2. Methodology - This section presents the basic components of the analysis procedure to be applied to the subject facility type. Included here are equations together with tabular and graphic information needed to complete an analysis.

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*Figures and Tables with hyphenated numbers are from Highway Capacity Manual, Special Report 209, Transportation Research Board, Washington, D.C. (1985).

3. Procedures for Application - Step-by-step instructions for applying capacity analysis computations are included in this section. Procedures are specified for operational analysis, design, and planning where appropriate. Worksheets are provided for most computational procedures, and explained in detail.

4. Sample calculations - This section presents a variety of example applications showing all computations required for analysis, and detailed discussions of results and interpretations. Sample calculations are provided for the full range of potential applications in each chapter.

The organization of the procedural chapters in this fashion will allow frequent users to focus on step-by-step instructions, without having to read or scan an entire chapter.

As an additional convenience for frequent users, most chapters contain an appendix in-which many figures and worksheets are reproduced for use, without the need to flip through pages of the manual text.

## DEFINITIONS and CONCEPTS

### Types of Facilities

Facilities may generally be classified into one of two categories: 1) uninterrupted flow, or 2) interrupted flow.

Note that this classification is for facility type, not flow conditions. Thus, a freeway at Level of Service F is still "uninterrupted flow." Capacity analysis for uninterrupted flow facilities must deal only with the physical constraints of the facility and traffic interactions. For interrupted flow facilities, the task is much more difficult, as signal systems place limitations on the times during which a physical facility is available for various traffic movements.

### Capacity and Level of Service Concepts

#### Capacity

The 1985 HCM defines capacity as "...the maximum rate of flow that can reasonably be expected to pass a point or uniform section of a lane or roadway under prevailing roadway, traffic, and control conditions."

There is only one major difference between this definition and that of the 1965 HCM. Most analyses in the 1985 HCM are based upon rates of flow for the peak 15-minute period within the hour of interest, usually the peak hour. Full-hour volumes are only used in a few exceptional cases, where the analyst is given an option of whether to use hourly volumes or flow rates. This permits the identification of short periods of capacity or over saturated operation, which may leave congestion taking much longer to dissipate.

**Level of Service**

The definition of Level of Service is also a familiar one. A level of service is defined as "...a qualitative measure describing operational conditions within a traffic stream, or their perception by motorists and/or passengers ...", with these conditions generally described by "...such measures as speed and travel time, freedom to maneuver, traffic interruptions, comfort and convenience, and safety."

While the definition of Level of Service is virtually the same as that used in the 1965 HCM, its implementation in procedures has been altered considerably. The one traffic parameter not mentioned in the definition of the Level of Service concept is volume. Motorists in a traffic stream do not perceive volume, as it is a point measure. They do perceive speed and other time-related measures, and they do perceive the proximity of other vehicles, which is related to density. Freedom to maneuver is also related to traffic density. Despite this, the methodologies of the 1965 HCM often used volume as a defining parameter for levels of service. Wherever possible, the 1985 HCM abandons this practice. Table 1-2, modified to include 1965, summarizes and compares the measures of effectiveness used to define levels of service in the 1965 and 1985 HCM's.

Note that it is only the cases of Ramp Junctions and Unsignalized Intersections that volume or volume-based parameters are used to DEFINE Level of Service. It should also be noted that while quality measures are used to DEFINE levels of service, these definitions are then related to volume levels which can be sustained at each level. Thus, the engineer is still able to work with volume or rate of flow as the primary parameter in capacity analysis.

TABLE 1-2 (MODIFIED)

COMPARATIVE LEVEL OF SERVICE MEASURES
IN THE 1965 AND 1985 HCM's

| TYPE OF FACILITY | MEASURE OF EFFECTIVENESS | |
|---|---|---|
| | 1965 HCM | 1985 HCM |
| Basic Freeway Sections | Operating Speed (mph) v/c Ratio | Density (pc/mi/ln) |
| Weaving Sections | Operating Speed (mph) | Avg. Travel Speed (mph) |
| Ramp Junctions | Merge/Diverge Vols. (vph) | Merge/Diverge Vols. (vph) |
| Multilane Highways | Operating Speed (mph) v/c Ratio | Density (pc/mi/ln) |
| Two-Lane Rural H'ways | Operating Speed (mph) | Percent Time Delay |
| Unsignalized Int. | * | Reserve Capacity (vph) |
| Signalized Int. | Load Factor | Delay (secs/veh) |
| Arterials | Avg. Travel Speed (mph) | Avg. Travel Speed (mph) |
| Transit | * | Load Factor (pers/seat) |
| Pedestrians | * | Space (sq ft/ped) |

* Not included in the 1965 HCM.

The familiar structure of six levels of service, A through F, remains in the new HCM. Some of the specific designations, however, have different meanings than in the past, and some care will be needed in interpreting results and presenting them to decision-makers over the next several years.

## Basic Principles of Traffic Flow

The operational state of any given traffic stream is defined by three primary measures:

1. Speed
2. Volume and/or Rate of Flow.
3. Density.

### Speed

The speed of each individual vehicle in a traffic stream varies. Any given traffic stream contains a range and distribution of individual vehicle speeds. To characterize a given traffic stream, a representative value must be taken. The 1985 HCM uses average travel speed as the major parameter for speed. Average travel speed is the length of the highway segment divided by the average total time of vehicles traversing the section.

### Volume and/or Rate of Flow

Volume is the total number of vehicles passing a point or section of a roadway during a specified time period. Rate of flow is an equivalent hourly rate at which vehicles pass a point or section of roadway for some period of time less than one hour. While the 1965 HCM based most analyses on full-hour volumes, the 1985 HCM bases most analyses on rates of flow for the peak 15 minutes within the hour of interest.

An example will illustrate the difference between these 2 terms. The following traffic comments were made during an hour-long study period:

| time period | volume for period (veh.) | rate of flow (vph) |
|---|---|---|
| 5:00-5:15 | 1,000 | 4,000 |
| 5:15-5:30 | 1,200 | 4,800 |
| 5:30-5:45 | 1,100 | 4,400 |
| 5:45-6:00 | 1,000 | 4,000 |
| 5:00-6:00 | 4,300 = full hour volume | |

The total volume for the hour is 4300 vehicles. The rate of flow, however, varies within each 15 minute period. During the 15-minute period of maximum flow, the rate of flow is 1200 vehicles - 0.25 hour, or 4800 vehicles/hour.

Peak rates of flow are related to hourly volumes through the peak hour factor (PHF), which is defined as the ratio of total hourly volume to the maximum 15-min rate of flow within the hour:

$$PHF = \frac{V}{4 \times V_{15}}$$

For our example the PHF is:

$$PHF = \frac{4{,}300}{4 \times 1200} = 0.90$$

The Peak Hour Factor is a measure of flow variation within a peak hour. Low values suggest much variation of flow, while values approaching 1.00 suggest very little variation of flow within the hour.

**Density**

Density is a measure of the number of vehicles occupying a given length of lane or roadway, and is measured in units of veh/mi or veh/mi/ln. It is difficult to measure directly in the field, unless elevated vantage points are available and photographic techniques are used. It is most often computed from the average travel speed and the rate of flow using the basic relationship:

Rate of Flow = Speed X Density

## Characteristics of Uninterrupted Flow

Figure 1-1 shows the general form of the relationships among speed, flow, and density which form the basis for capacity analysis of uninterrupted flow facilities. As illustrated in Figure 1-1, any rate of flow other than capacity can occur under two different conditions -- one with a high speed and low density, the other with high density and low speed. The entire high-density, low-speed side of the curves is considered to be unstable, and represents forced, or breakdown flow. The low-density, high-speed side is the stable flow region -- the focus of capacity analysis. Levels-of-Service A through E are defined on the stable side of the curves, with maximum flow boundary of LOS E placed at capacity for uninterrupted flow.

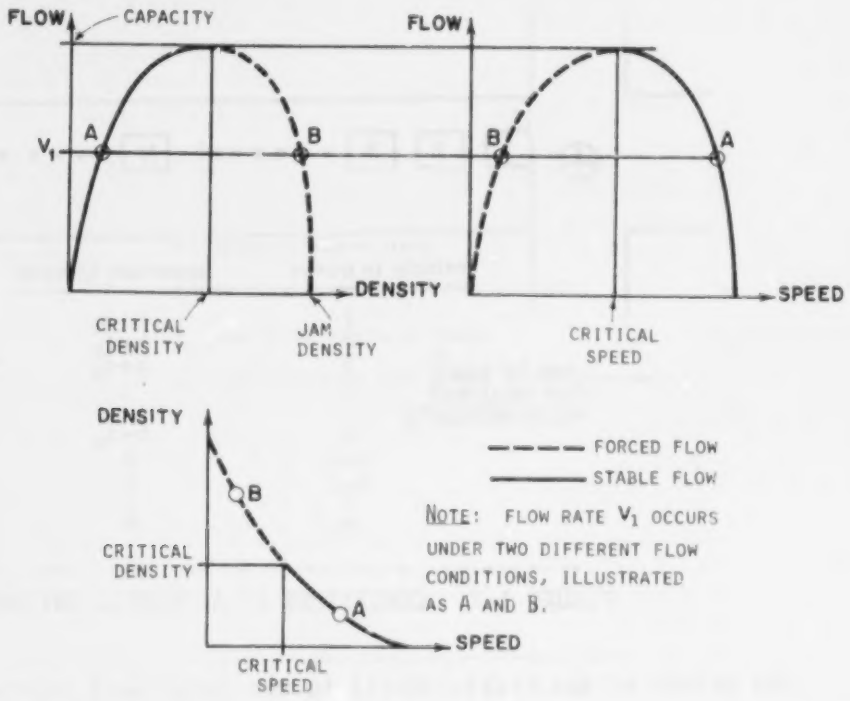



*Figure 1-1. Relationships among speed, density, and rate of flow on uninterrupted flow facilities.*

## Characteristics of Interrupted Flow

Interrupted flow is far more complex than uninterrupted flow. The principal complicating factor for interrupted flow facilities is that roadway space is not available at all times for all traffic movements. Traffic signals and other interruptions control the portion of time that space may be used by various movements. Thus, unlike uninterrupted flow, where capacity is based solely on geometrics and traffic conditions, for interrupted flow it is also based on the control of time. Thus, capacity is far more variable and

subject to the intervention of the traffic engineer. At signalized intersections, traffic on all lanes will be periodically stopped. When the signal turns green, the dynamics of starting a standing queue of vehicles must be considered.

Figure 1-2 illustrates a queue of vehicles stopped at a signal. When the signal turns green, the queue begins to move. The headways between vehicles can be observed as they cross the curb line of the intersection.

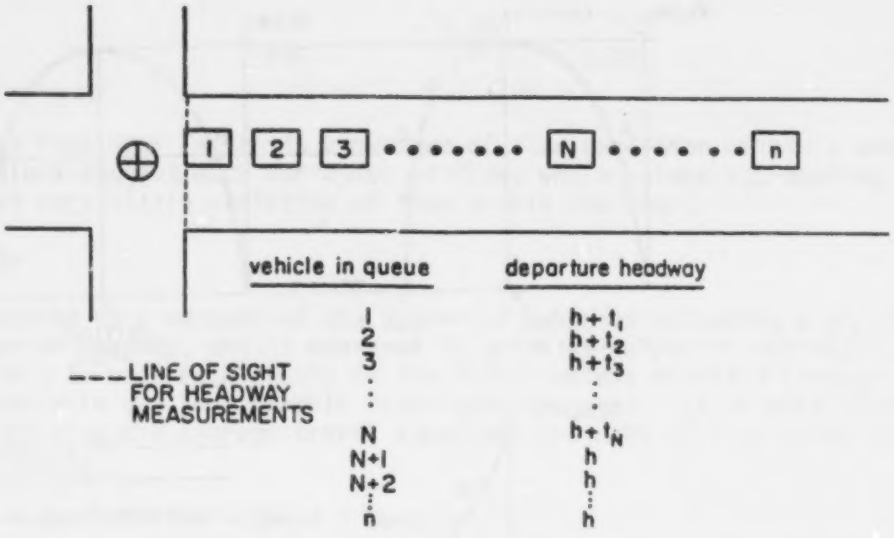



FIGURE 1-2. CONDITIONS AT A TRAFFIC INTERRUPTION.*

The driver of the first vehicle in the queue must observe the signal change to green and react to it before accelerating through the intersection. The first headway will be longer as a result of this process. After some number of vehicles, N, the effect of the start-up reaction and acceleration has dissipated. The headway for subsequent vehicles will be relatively constant.

In figure 1-2, this constant average headway is, "h", and is achieved after "N" vehicles. The headways for the first N vehicles are greater than h.

Figure 1-3 shows a conceptual plot of headways. For purposes of illustration only, N is assumed to equal 6, i.e., the start-up and acceleration increment disappears after the 6th vehicle. The value "h" is defined as the saturation headway and it is estimated as the constant average headway between vehicles which occurs after the N-th vehicle in the queue and continues until the last vehicle in the initial queue clears the intersection.

Saturation flow is defined as the flow rate per lane at which vehicles can pass through a signalized intersection in such a stable moving queue. The saturation flow rate represents the number of vehicles per hour per lane that can pass through an intersection if the green signal were available for the full hour, and the flow of vehicles were never halted. The reality of flow at a signalized intersection is that flow is periodically halted.

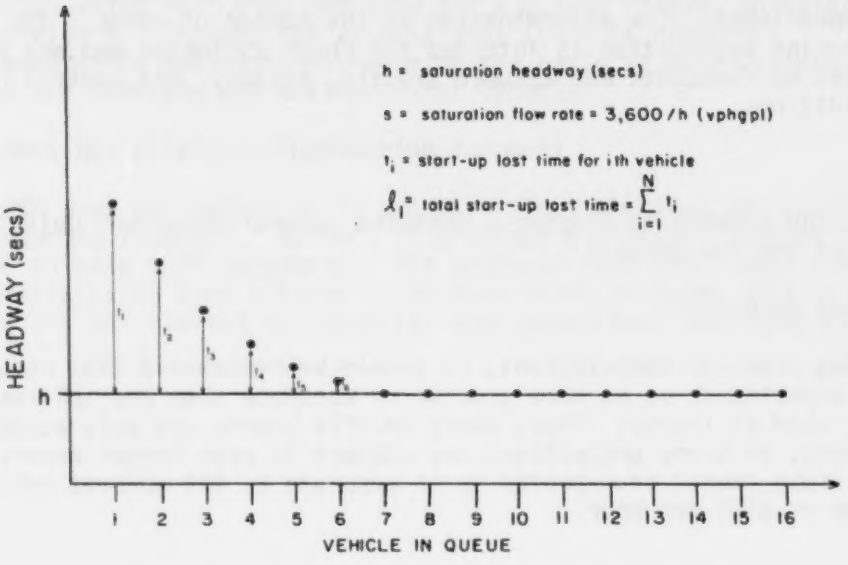



FIGURE 1-3. SATURATION FLOW RATE AND LOST TIME.

## APPLICATIONS

### Levels of Analysis

Most of the procedural chapters address three different computational applications: operational analysis, design, and planning.

1. Operational analysis is the most detailed and flexible application of capacity analysis techniques. It allows for an evaluation of level of service on an existing facility or what would result from alternative spot and section improvements to the facility. Alternative designs for new facilities can be evaluated in a simular manner.

2. The design application determines the number of lanes required to provide for a specified level of service, as well as assessing the impact of other geometric design elements. Design computations are generally limited in scope, and often result in alternatives that are subjected to detailed operational analysis.

3. Planning computations have the same objective as design computations: The determination of the number of lanes. The planning application is intended for rough estimates and are often based on forecasts and assumed traffic, roadway, and control conditions.

### Summary

The closing summary of Chapter 1 contains several important guidelines on the use of the new manual:

#### Precision and Accuracy

In making capacity computations, it should be remembered that results can be expected to be no more precise or accurate than the information or data used as inputs. Thus, where traffic counts are only accurate to +5 percent, or where projections are subject to even larger errors, computations cannot be expected to be accurate to the nearest vehicle per hour or mile per hour.

#### Field Data

The basic traffic data required to conduct any level of analysis are volume and traffic characteristics. Procedures of this manual have been calibrated to estimate performance parameters such as speed, density, delay and others, based on specified volumes and traffic characteristics. Where local data are available in sufficient quantity, and in an acceptable form, they may be used to "fine-tune" the manual procedures. The results of capacity computations will not be more accurate than the input data on which whey were based.

**The HCM Is Not A Standard**

The HCM is not a standard. The Transportation Research Board Committee on Highway Capacity and Quality of Service is not a standard-making body. The HCM defines levels of service merely as descriptors. It does not recommend minimum levels of acceptability.

**The HCM Is A Tool For Alternative Analysis**

The most potent use of capacity analysis procedures is in alternative analysis. It gives the engineer the capability to evaluate many different alternative solutions to a given problem by revising geometric, traffic, and control parameters. With the addition of microcomputer software, the engineer will be freed from much of the computational drudgery of procedures, and will be freer to create and test more design and control options for a given problem.

**The HCM Does Not Mandate Change**

A solution generated by capacity analysis does not mandate implementation of any particular plan or design. The HCM may yield a solution which indicates the need for a 6-lane freeway to maintain level of service D. It does not dictate that LOS D is required. Implementation decisions are made on the basis of many factors, among which are economic and environmental impacts.

**The HCM Does Not Eliminate Engineering Judgment**

The HCM does not reduce or in any way affect the need for application of engineering judgment. Rather, it provides additional information upon which to base such judgments. The engineer must still make complex judgments based upon a range of factors which includes, but is most certainly not limited to, capacity and operational considerations.

## CHAPTER 2: TRAFFIC CHARACTERISTICS

Chapter 2 of the 1985 HCM provides a review of various traffic characteristics which affect or form the basis of capacity analysis. Included in the chapter are:

1. Maximum Observed Volumes and Flow Rates

2. Volume Characteristics

   a. Temporal Variations
   b. Spatial Variations
   c. Traffic Composition
   d. Weather Impacts

3. Speed Characteristics

4. Speed-Flow-Density Relationships for Uninterrupted Flow

5. Spacing and Headway Characteristics

6. Saturation Headways and Lost Times Under Interrupted Flow

Every procedure in the manual is based upon some calibrated set of basic traffic characteristics for a particular type of facility. These, in general, represent reasonable national averages, which may vary from place to place with local driving habits. Chapter 2 gives an informational appreciation of the kinds and degrees of variation that can be expected over a range of common traffic characteristics, and describes the relationships between these characteristics and capacity of traffic facilities.

## CHAPTER 3: BASIC FREEWAY SEGMENTS

Chapter 3 of the HCM is one of four chapters in the new HCM dealing with freeways and their component subsections. Chapter 3 deals with the basic freeway section, i.e., sections of the freeway outside the influence of turbulence from merging, diverging, or weaving movements.

While the basic form of computational procedures for basic freeway sections is similar to those used in the 1965 HCM, there are a number of significant differences in values used and in interpretations of results. The most central of these changes is in the base relationships among speed, flow, and density calibrated for the average North American freeway operating under ideal conditions. These are shown in Figures 3-4 and 3-3.

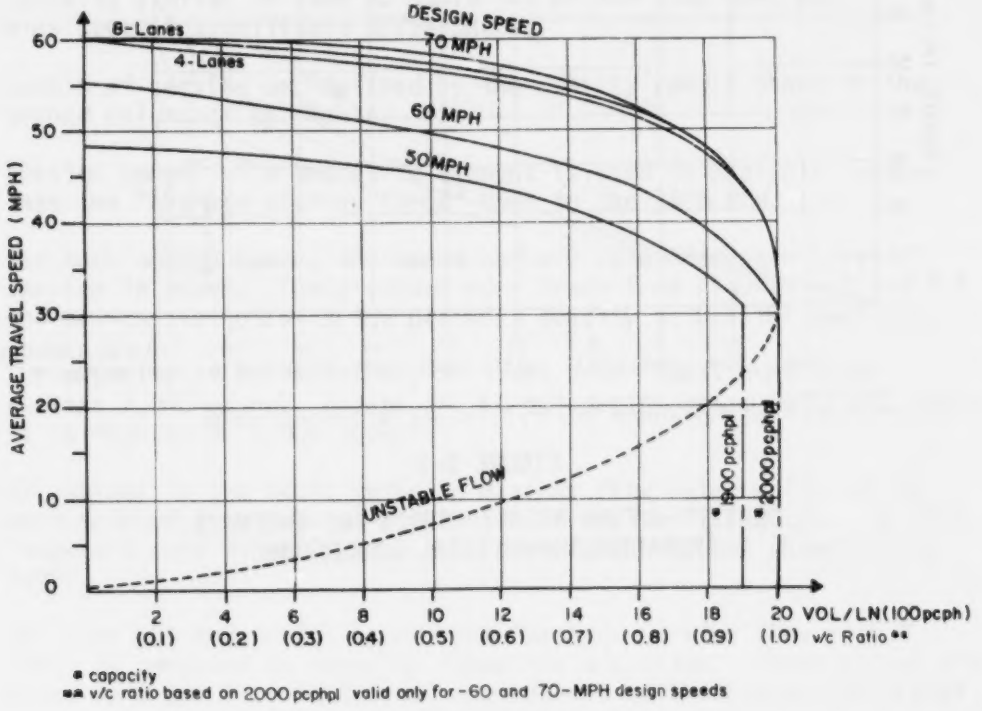
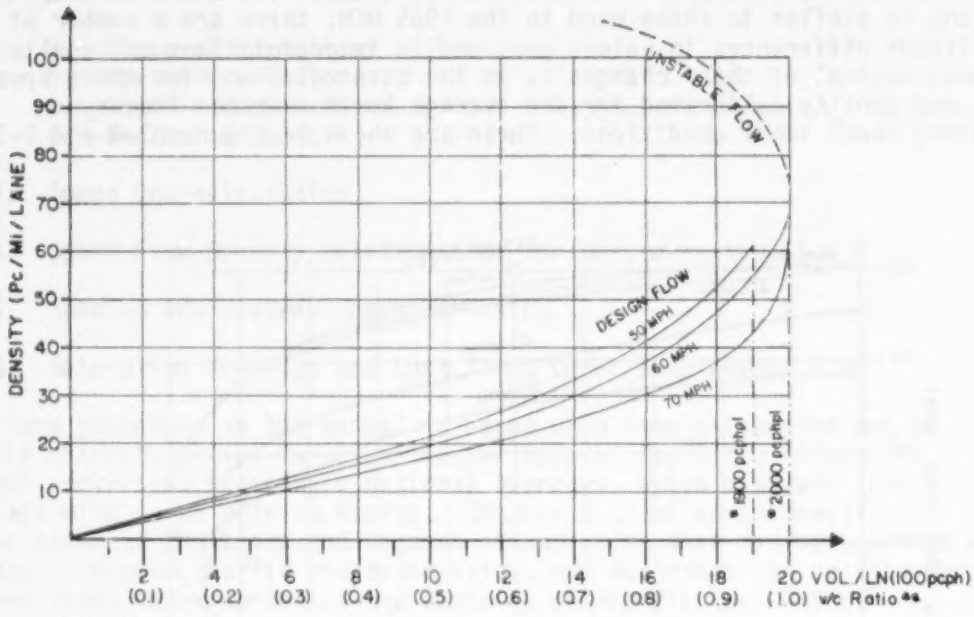



FIGURE 3-4

SPEED - FLOW RELATIONSHIP FOR FREEWAYS
OPERATING UNDER IDEAL CONDITIONS

Note that in Figure 3-4 for 70-mph design speed sections speed does not vary significantly with flow for rates of flow up to approximately 1,600 pcphpl. Thereafter, speed precipitously declines as flow approaches capacity. Because of this, speed could not be taken as a major indicator of level of service. In the entire range of flow from 0 to 1,600 pcphpl, speed varies only by approximately 70 mph. It would have been impossible, therefore, to define meaningful Level of Service related to such a narrow speed range.

This is not true for the relationship between density and flow, depicted in Figure 3-3. Density varies throughout the range of flows from 0 to 2,000 pcphpl, and is, therefore, more easily related to level of service.




* capacity
** v/c ratio based on 2000 pcphpl valid only for - 60 and 70 - MPH design speeds

FIGURE 3-3

DENSITY - FLOW RELATIONSHIPS FOR FREEWAYS
OPERATING UNDER IDEAL CONDITIONS

Because of these characteristics, density was chosen as the principal measure of effectiveness for use in defining freeway Levels of Service. Further, density relates to many characteristics clearly perceived by drivers, such as freedom to maneuver and comfort levels. Using the standard relationships of speed, flow, and density the base LOS table for freeways was constructed, Table 3-1.

## TABLE 3-1

## LEVEL OF SERVICE CRITERIA FOR BASIC FREEWAY SECTIONS

| LOS | DENSITY (PC/MI/LN) | 70 MPH DESIGN SPEED | | | 60 MPH DESIGN SPEED | | | 50 MPH DESIGN SPEED | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | SPEED[b] (MPH) | v/c | MSF[a] (PCPHPL) | SPEED[b] (MPH) | v/c | MSF[a] (PCPHPL) | SPEED[b] (MPH) | v/c | MSF[a] (PCPHPL) |
| A | ≤ 12 | ≥ 60 | 0.35 | 700 | — | — | — | — | — | — |
| B | ≤ 20 | ≥ 57 | 0.54 | 1,100 | ≥ 50 | 0.49 | 1,000 | — | — | — |
| C | ≤ 30 | ≥ 54 | 0.77 | 1,550 | ≥ 47 | 0.69 | 1,400 | ≥ 43 | 0.67 | 1,300 |
| D | ≤ 42 | ≥ 46 | 0.93 | 1,850 | ≥ 42 | 0.84 | 1,700 | ≥ 40 | 0.83 | 1,600 |
| E | ≤ 67 | ≥ 30 | 1.00 | 2,000 | ≥ 30 | 1.00 | 2,000 | ≥ 28 | 1.00 | 1,900 |
| F | > 67 | < 30 | [c] | [c] | < 30 | [c] | [c] | < 28 | [c] | [c] |

[a] Maximum service flow rate per lane under ideal conditions.
[b] Average travel speed.
[c] Highly variable, unstable.
NOTE: All values of *MSF* Rounded to the nearest 50 pcph.

The table is similar in form to Table 9-1 of the 1965 HCM, but incorporates several significant differences.

1. Levels of service are defined by the density ranges shown in the second column of the table.

2. "Design speed" of a geometric element is used in analysis, rather than the "average highway speed" used in the 1965 HCM.

3. For each design speed, the speed and v/c value for each level of service is shown. These values were drawn from Figures 3-3 and 3-4 and are consistent with the defining density values for each LOS,

4. The capacity of basic freeway sections under ideal conditions remains 2,000 pcphpl, except for 50-mph design speed sections, where it is reduced to 1,900 pcphpl.

5. All values in the table refer to maximum flow rates which can be accommodated in a peak 15-minute period for the given LOS. As flow rates are used directly, the peak hour factor is not shown in the table.

6. For each LOS and design speed, the "maximum service flow rate," (MSF) is computed as capacity times the v/c ratio. These values are given on a per lane basis, and are rounded to the nearest 50 pcphpl.

The values in Table 3-1 are for ideal conditions, which include 12-ft lane widths, 6-ft minimum lateral clearances, 70-mph design speed, and all passenger cars in the traffic stream. They must be adjusted to reflect prevailing roadway, traffic, and control conditions. There are two analytic relationships used to accomplish this.

$$SF_i = MSF_i \times N \times f_w \times f_{HV} f_p$$

$$MSF_i = c_j \times (v/c)_i$$

These are combined to form a single equation which is most often used in computational procedures:

$$SF_i = c_j \times (v/c)_i \times N \times f_w \times f_{HV} \times f_p$$

Where: $SF_i$ = total service flow rate for LOS i, under prevailing roadway and traffic conditions, for one direction.

$c_j$ = capacity per lane for freeway of design speed j.

$(v/c)_i$ = maximum v/c ratio which can be accommodated at level of service i.

N = number of lanes in one direction.

$f_w$ = adjustment factor for the effect of narrow lanes and inadequate lateral clearances.

$f_{HV}$ = adjustment factor for the effect of trucks, buses, and recreational vehicles in the traffic stream.

$f_p$ = adjustment factor for the effect of different driver populations. (NEW FOR 1985 HCM.)

This equation is used in all computational analyses. In "operational analysis," where an existing or proposed condition is solved for a prediction of level of service and likely operating conditions, the equation is solved for v/c, using the existing or forecast flow rate for SF. In "design analysis," the equation is solved for N, the number of lanes needed to provide given or forecast flow rate with a prescribed level of service. In "planning analysis," the number of lanes are also determined, but using approximations or forecasts for input values.

The adjustment factor for lane width and lateral clearance, $f_w$, is the same as in the 1965 HCM. There has been no new research in this area, and no revisions have been made.

The adjustment factor for heavy vehicles, $f_{HV}$, reflects considerable new knowledge acquired since the 1965 HCM. The heavy vehicle factor is computed from passenger car equivalent values for three categories of vehicles: trucks, buses, and recreational vehicles. Values represent the number of passenger cars which would occupy as much space as one truck, bus, or recreational vehicle under the given roadway and traffic conditions.

On specific upgrades, analysts may categorize the characteristics of the truck population as light, typical, or heavy. Only one value is used, rather than dividing the truck population into three categories. Passenger car equivalents for the typical truck reflect the improved operating characteristics of such vehicles and are in general, less than the values of the 1965 HCM.

The consideration of recreational vehicles as a separate category is also new in the 1985 HCM, and reflects a growing population of such vehicles and their unique characteristics.

The adjustment factor for driver population, $f_p$, reflects research data showing that from 10% - 25% of capacity may be lost when comparing commuter or regular traffic flow to weekend or recreational traffic. While this new factor requires judgment, and precise values are not given, it serves as a reminder to consider this important factor in all analyses. In most analyses, the factor will be regular or commuter traffic.

Chapter 3 of the 1985 HCM presents a familiar computational framework with some new concepts built in. Like all computational chapters of the new manual, it includes worksheets and sample calculations to aid the user in becoming familiar with procedures.

## CHAPTER 4: WEAVING AREAS

The weaving area analysis procedure of the 1985 HCM is significantly different from any of the procedures previously published in either the 1965 HCM, TRB Circular 212, or various research documents. It is based upon an algorithm developed by JHK & Associates for prediction of weaving and non-weaving vehicle speeds through a weaving area.

The procedure deals with three geometric parameters, each of which influences lane-changing behavior in a weaving section.

1. Length
2. Number of Lanes
3. Configuration

The last of these is new, and is perhaps the most critical of the three. Configuration relates to the manner in which entry and exit lanes to and from the section line up with one another. This, in turn, controls the number of lane changes which weaving vehicles must make to successfully complete a weaving maneuver. The three basic weaving configuration types identified in the 1985 HCM are shown on the following page.

In Type A sections,--also called a 1-sided weave or ramp weave--weaving vehicles in each direction must make one lane change. All of these lane changes must cross a single lane marking, referred to as the "crown line." In Type B sections, one weaving movement can be made without any lane changes, while the reverse movement requires one lane change. In Type C sections, one weaving movement is made without requiring any lane changes, but the reverse weaving movement requires a minimum of two lane changes. Thus, for any given length and width of weaving section, there are three different configurations which can be provided, each producing markedly different operating conditions. The entire procedure of the 1985 HCM is stratified on the basis of configuration type.

Operating conditions in weaving areas are further stratified as being "unconstrained," or "constrained." In constrained operation, weaving vehicles are restricted by configuration to a smaller portion of the roadway than they would normally occupy. The result of this is that weaving vehicles may be subject to congestion, while non-weaving vehicles experience markedly higher speeds and better levels of service. In unconstrained operation, weaving and non-weaving vehicles reach a competitive balance of space utilization such that all vehicles experience similar speeds and levels of service.

Computations involve three primary steps. The first of these is the estimation of the average travel speed of weaving ($S_w$) and non-weaving ($S_{nw}$) vehicles in the weaving area. Unconstrained operation is initially assumed for this computation. Table 4-3 shows the equations used for this purpose. The chapter is built around these 12 equations. The equations are asymptotic to 15 mph and 65 mph.

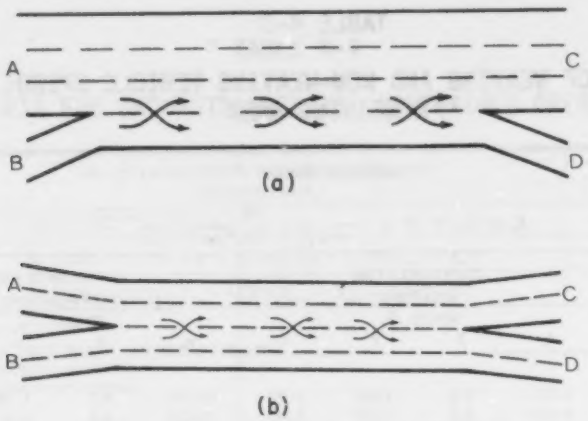



*Figure 4-3. Type A weaving areas: (a) ramp-weave/one-sided weave, and (b) major weave with crown line.*

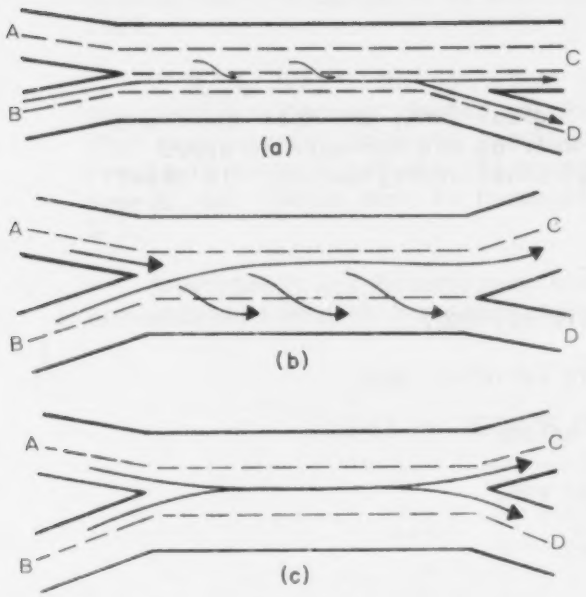



*Figure 4-4. Type B weaving areas: (a) major weave with lane balance at exit gore, (b) major weave with merging at entrance gore, and (c) major weave with merging at entrance gore and lane balance at exit gore.*

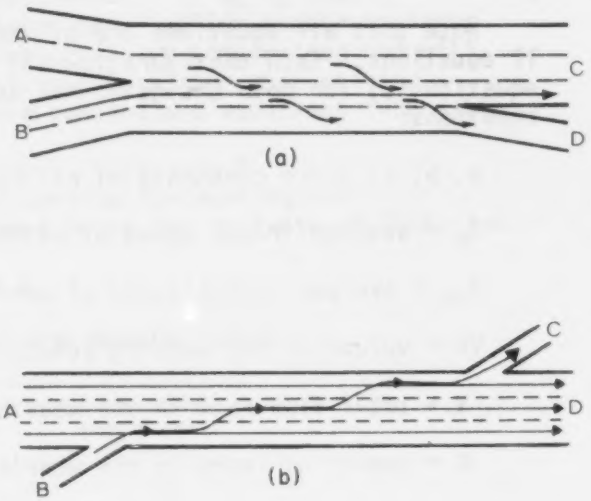



*Figure 4-5. Type C weaving areas: (a) major weave without lane balance or merging, and (b) two-sided weave.*

**FIGURES 4-3, 4-4, AND 4-5**

**WEAVING AREA CONFIGURATION TYPES**

## TABLE 4-3

## COMPUTATION OF WEAVING AND NON-WEAVING VEHICLE SPEEDS THROUGH A WEAVING AREA

GENERAL FORM:

$$S_w \text{ or } S_{nw} = 15 + \frac{50}{1 + a(1 + VR)^b (v/N)^c / L^d}$$

| TYPE OF CONFIGURATION | CONSTANTS FOR WEAVING SPEED, $S_w$ a | b | c | d | CONSTANTS FOR NONWEAVING SPEED, $S_{nw}$ a | b | c | d |
|---|---|---|---|---|---|---|---|---|
| TYPE A | | | | | | | | |
| Unconstrained | 0.226 | 2.2 | 1.00 | 0.90 | 0.020 | 4.0 | 1.30 | 1.00 |
| Constrained | 0.280 | 2.2 | 1.00 | 0.90 | 0.020 | 4.0 | 0.88 | 0.60 |
| TYPE B | | | | | | | | |
| Unconstrained | 0.100 | 1.2 | 0.77 | 0.50 | 0.020 | 2.0 | 1.42 | 0.95 |
| Constrained | 0.160 | 1.2 | 0.77 | 0.50 | 0.015 | 2.0 | 1.30 | 0.90 |
| TYPE C | | | | | | | | |
| Unconstrained | 0.100 | 1.8 | 0.80 | 0.50 | 0.015 | 1.8 | 1.10 | 0.50 |
| Constrained | 0.100 | 2.0 | 0.85 | 0.50 | 0.013 | 1.6 | 1.00 | 0.50 |

Note that all equations are of the same logical form, and that there are 12 equations. Each configuration (3) has a weaving and non-weaving speed equation (2) for both unconstrained and constrained operation (2). In these equations:

a, b, c, d are constants of calibration.

$S_W$ = average travel speed of weaving vehicles, mph.

$S_{nw}$ = average travel speed of non-weaving vehicles, mph.

VR = volume ratio; weaving volume/total volume.

V = total flow rate in the weaving area, vph.

N = number of lanes in the weaving area.

L = length of the weaving area, ft.

After the initial computation of weaving and non-weaving vehicle speeds based upon assumed unconstrained operation, this assumption must be checked. Table 4-4 shows equations for accomplishing this.

## TABLE 4-4

## CRITERIA FOR UNCONSTRAINED VS. CONTRAINED OPERATION

| TYPE OF CONFIGURATION | NO. OF LANES REQ'D FOR UNCONSTRAINED OPERATION, $N_w$ | MAX. NO. OF WEAVING LANES, $N_w$ (max) |
|---|---|---|
| Type A | $2.19\ N\ VR^{0.571}\ L_H^{0.234}/S_w^{0.438}$ | 1.4 |
| Type B | $N\ \{0.085 + 0.703\ VR + (234.8/L) - 0.018(S_{nw} - S_w)\}$ | 3.5 |
| Type C | $N\ \{0.761 - 0.011\ L_H - 0.005(S_{nw} - S_w) + 0.047\ VR\}$ | 3.0 [b] |

[a] All variables are as defined in Table 4-2.
[b] For 2-sided weaving areas, *all* freeway lanes may be used as weaving lanes.
NOTE: When $N_w \leq N_w$ (max), operation is unconstrained.
When $N_w > N_w$ (max), operation is constrained.

For each type of configuration, an equation calculates $N_W$, the number of lanes needed by weaving vehicles to achieve unconstrained operation. This result is compared to the tabulated value of $N_W$ (max), the maximum number of lanes weaving vehicles may occupy for any given configuration, determined from analysis of weaving area research studies conducted since 1963.

If $N_W \leq N_W$ (max), the operation is unconstrained. If $N_W > N_W$ (max), the configuration constrains weaving vehicles to a smaller number of lanes than required for balanced operation. Such cases are constrained and will result in average non-weaving speeds significantly higher than weaving speeds, and speeds must be recomputed using constrained equations from Table 4-3.

Once speeds are determined, levels of service for weaving and non-weaving vehicles are separately assigned using the criteria of Table 4-6.

## TABLE 4-6

## LEVEL OF SERVICE CRITERIA FOR WEAVING AREAS

| LEVEL OF SERVICE | MIN. AVG. WEAVING SPEED, $S_w$ (MPH) | MIN. AVG. NON-WEAVING SPEED, $S_{nw}$ (MPH) |
|---|---|---|
| A | 55 | 60 |
| B | 50 | 54 |
| C | 45 | 48 |
| D | 40 | 42 |
| E | 35/30 [a] | 35/30 [a] |
| F | < 35/30 [a] | < 35/30 [a] |

[a] The 35-mph boundary for LOS E/F is used when comparing to computed speeds using the equations of Table 4-3. The 30-mph boundary is used for comparison to field-measured speeds.

In addition to the computational procedure, the 1985 HCM prescribes limiting values for total weaving volume, total per lane volume, volume ratio. and length of weaving areas which are of great value in considering potential configurations. Procedures for multiple weaving sections are also prescribed, and are far simpler than those of the 1965 HCM. Sample calculations are include to assist in the understanding of the procedures.

## CHAPTER 5: RAMPS AND RAMP JUNCTIONS

Chapter 5 treats the capacity characteristics of freeway merging and diverging at ramps, as well the characteristics of ramp roadways themselves. The methodology is very similar to that of the 1965 HCM, as the basic procedures remain unchanged. No major new research has been conducted since the 1965 HCM to allow major changes in the way merging and diverging analyses take place. While the basic procedures are those of the 1965 HCM, research has allowed a number of critical modifications to be made. The four key differences and their impact on analyses are:

1. The 1985 HCM eliminates the dual procedures of the 65 HCM. The nomograph procedure, the A-C procedure of the 1965 HCM, was in fact calibrated for the full range of levels of service. The 1985 HCM adopts the nomograph procedure for all cases where a nomograph is available.

2. Five of the nomographs of the 1965 HCM dealt with on-ramp, off-ramp sequences connected by a continuous auxiliary lane. These are now treated as weaving areas, and are deleted from the 1985 HCM, reducing the number of nomographs to 13.

3. The 1965 HCM used a base of 5% trucks in all tables and figures. The 1985 HCM converts all base tables and figures to pcph, or 0% trucks, for consistency with other chapters and ease of computations.

4. Level of service criteria for merging and diverging have been revised, and the level of service check for weaving has been eliminated. Weaving configurations are now handled by weaving area analysis procedures of Chapter 4.

Table 5-1 depicts the revised level of service criteria for merge, diverge, and freeway flows. The total freeway flow criteria are merely transcribed from Chapter 3 for convenience. They are the same as the criteria for basic freeway sections shown previously. For any given level of service, it is reasoned that permissible merge and diverge flow rates should be lower than the per lane service flow rate permitted on a basic freeway section. This accounts for the increased turbulence of merge and diverge maneuvers while maintaining essentially similar operating conditions to those described for basic freeway sections at the stated level of service. In general, merge areas will have higher levels of turbulence than diverge areas serving similar flows. Diverge service flow rates are, therefore, somewhat higher than merge service flow rates for each level of service.

## TABLE 5-1

## LEVEL OF SERVICE CRITERIA FOR RAMP JUNCTIONS

| LEVEL OF SERVICE | MERGE FLOW RATE (PCPH)[a] $v_m$ | DIVERGE FLOW RATE (PCPH)[b] $v_d$ | FREEWAY FLOW RATES (PCPH)[c], $v_f$ | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 70-MPH DESIGN SPEED | | | 60-MPH DESIGN SPEED | | | 50-MPH DESIGN SPEED | | |
| | | | 4-LANE | 6-LANE | 8-LANE | 4-LANE | 6-LANE | 8-LANE | 4-LANE | 6-LANE | 8-LANE |
| A | ≤ 600 | ≤ 650 | ≤ 1,400 | ≤ 2,100 | ≤ 2,800 | [d] | [d] | [d] | [d] | [d] | [d] |
| B | ≤ 1,000 | ≤ 1,050 | ≤ 2,200 | ≤ 3,300 | ≤ 4,400 | ≤ 2,000 | ≤ 3,000 | ≤ 4,000 | [d] | [d] | [d] |
| C | ≤ 1,450 | ≤ 1,500 | ≤ 3,100 | ≤ 4,650 | ≤ 6,200 | ≤ 2,800 | ≤ 4,200 | ≤ 5,600 | ≤ 2,600 | ≤ 3,900 | ≤ 5,200 |
| D | ≤ 1,750 | ≤ 1,800 | ≤ 3,700 | ≤ 5,550 | ≤ 7,400 | ≤ 3,400 | ≤ 5,100 | ≤ 6,800 | ≤ 3,200 | ≤ 4,800 | ≤ 6,400 |
| E | ≤ 2,000 | ≤ 2,000 | ≤ 4,000 | ≤ 6,000 | ≤ 8,000 | ≤ 4,000 | ≤ 6,000 | ≤ 8,000 | ≤ 3,800 | ≤ 5,700 | ≤ 7,600 |
| F | WIDELY VARIABLE | | | | | | | | | | |

[a] Lane-1 flow rate plus ramp flow rate for one-lane, right-side on-ramps.
[b] Lane-1 flow rate immediately upstream of off-ramp for one-lane, right-side ramps.
[c] Total freeway flow rate in one direction upstream of off-ramp and/or downstream of on-ramp.
[d] Level of service not attainable due to design speed restrictions.

Computational procedures are similar to those of the 1965 HCM. The sequencing of computations is important, however, and steps must be followed in a precise order to avoid serious error:

1. Establish the ramp configuration with respect to size of freeway and location of adjacent upstream and downstream ramps. Establish demand volumes and traffic characteristics at the ramp in question, and at adjacent ramps. Establish all other geometric conditions, such as lane widths, lateral clearances, grades, etc.

2. Select a nomograph appropriate to the configuration under study, or if none is available, use the general procedure. Using the selected method, compute Lane 1 volume immediately upstream of the ramp in question. This is done BEFORE converting volumes to pcph or to flow rates.

3. Find the percentage of trucks (or other heavy vehicles) in Lane 1, using relationships provided for this purpose.

4. Convert all ramp, Lane 1, and freeway volumes to pcph, using passenger car equivalents for basic freeway sections, given in Chapter 3.

5. Compute checkpoint volumes for all merge, diverge, and freeway points at which level of service is to be checked.

6. Convert merge, diverge, and freeway checkpoint volumes to rates of flow by dividing by the peak hour factor.

7. Compare merge, diverge, and freeway checkpoint flow rates to the criteria shown in Table 5-1.

The sequence is most important. Even though it would seem easier (computationally) to convert all volumes to pcph and flow rates as a first step, this would be incorrect. All nomograph relationships were calibrated with mixed vph and full-hour volumes, and must be used in a consistent manner.

In addition to the revisions in basic procedures, the 1985 HCM adds material not present in previous editions to allow for a broader range of applications. Among the important additions are:

1. Provisions for analysis of ramp junctions on freeways with 5 lanes in one direction.

2. Provisions for analysis of left-side ramp junctions.

3. Criteria for service flow rates on ramp roadways themselves.

4. Discussion of the impact of ramp geometry on merge and diverge performance. No quantitative data are available.

5. Discussion of the impact of ramp control on merge and diverge operations. No quantitative data are available.

The 1985 HCM clarifies and simplifies the previous computational procedures for analysis of ramp-freeway junctions. although the basic methodology remains the same.

## CHAPTER 6: FREEWAY SYSTEMS

The chapter on "Freeway Systems" is new to the 1985 HCM. Its intent is to provide for a framework for integrated analysis of freeways consisting of various basic, weaving, merging, and diverging sections. The chapter also covers several special topics which can affect freeway operations in general, such as work zones, weather, and similar factors. A brief summary of chapter contents is provided by subject area.

### Combined Analysis of Freeways

This material provides a framework for the design or operational analysis of a lengthy freeway facility which includes basic sections, weaving sections, and ramp junctions. It provides guidelines for:

1. Identification of subsections requiring separate capacity analysis.

2. Impact ranges for weaving, merge, and diverge areas.

3. Computational sequences for design and operational analysis.

4. Interpretation of results for overlapping segments.

5. Generation and analysis of alternative configurations.

Figure 6-5, for example, shows a graphic technique for displaying and analyzing levels of service over several adjacent freeway sections. The graphic is based upon influence ranges for ramp junctions and weaving areas. On-ramps influence level of service of 500 ft upstream and 2,500 ft downstream of the junction; off-ramps influence level of service for 2,500 ft upstream and 500 ft downstream of the junction; weaving areas influence level of service for 500 ft upstream and downstream of the weaving area. These ranges of influence assume stable flow.

Graphic displays similar to that in Figure 6-5 can be used to identify critical sections restraining capacity, and can serve to focus attention on areas needing improvement.

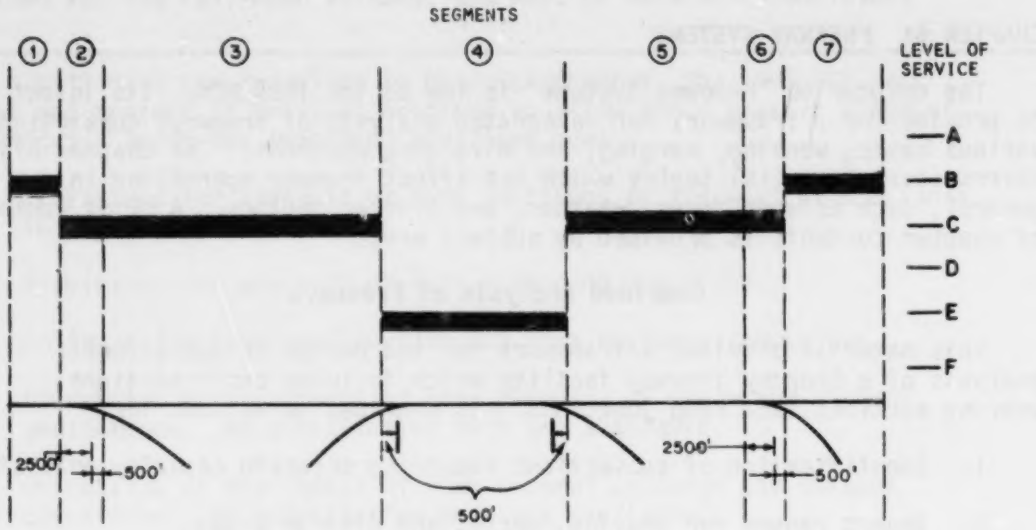



**FIGURE 6-5**

**GRAPHIC PRESENTATION OF FREEWAY SYSTEM LEVEL OF SERVICE**

### Analysis of Breakdown Conditions

Chapter 6 of the HCM also contains a set of procedures for the analysis of queue build-ups and dissipation as a result of breakdowns, or the occurrence of level of service F. Procedures are based upon a deterministic analysis of queueing, and rely on graphic plots of bottleneck capacity vs. arrival flows over the period of the breakdown.

Several key points should be made as part of the analyses presented:

1. The capacity of a freeway facility has been established as 2,000 pcphpl. There is considerable evidence to suggest that this value is reached only from stable flow. Several studies indicate that once a breakdown, or level of service F, is established, the capacity which can be sustained is somewhat lower than 2,000 pcphpl, perhaps as low as 1,600-1,800 pcphpl. These studies indicate that full capacity, or 2,000 pcphpl, cannot be re-established until all queued vehicles have been cleared and stable flow has been restored. This phenomenum is due to the inability of vehicles leaving or passing through a standing queue to do so at the same rate as can be achieved in a stable, moving traffic stream. This property makes recovery from breakdowns a very difficult problem, and can extend congestion long beyond the time when arrival flows exceed the stable capacity of a bottleneck.

2. A freeway queue is a dynamic queue, i.e., vehicles continuously "shuffle" forward as vehicles discharge from the head of the queue. On the average, vehicles in such a queue are spaced at 40 to 50 ft apart. This is considerably more than the 20 to 25 ft consumed by each vehicle in a stopped queue at a red light. This causes freeway queues to propogate reasonably quick, and to extend for considerable distances where flows exceed capacity significantly.

3. Graphic plots of bottleneck capacity and arrival flows can be used to identify the maximum number of vehicles queued, the maximum time spent by a vehicle in the queue, the time required to dissipate the queue, and the total vehicle-hours of delay consumed by all vehicles in the queue.

Figure 6-6 shows a plot of capacity vs. arrival flow for a location at which an incident blocks one lane of a three-lane section for 15 minutes. The illustrative analysis shows how such a breakdown can cause queueing which does not dissipate until almost three hours later, despite the fact that the incident lasted only 15 minutes. The major reason for this is the loss in capacity once queues are established.

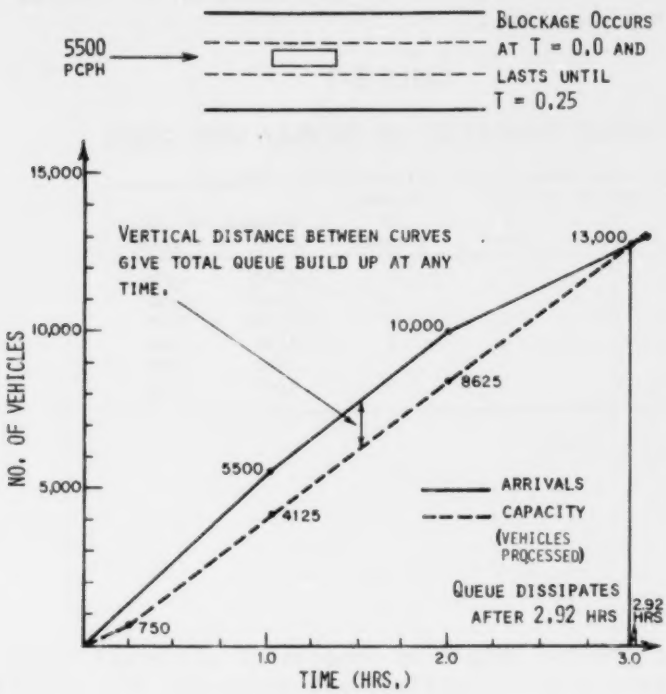



**FIGURE 6-6**

**GRAPHIC PRESENTATION OF QUEUEING FROM AN INCIDENT**

## Freeway Surveillance and Control

Chapter 6 of the 1985 HCM also contains a qualitative discussion of freeway surveillance and control systems, including ramp controls and their implications.

It is noted that there is no evidence that freeway surveillance and control systems increase the capacity of a freeway system. The primary objective of surveillance and control is the maintenance of capacity or near-capacity operations without disruption from incidents or other traffic breakdowns. Key components of surveillance and control programs include the speedy observation and clearance of accidents and other incidents, and the control or limitation of on-ramp flows to avoid exceeding the capacity of downstream sections.

## Capacity of Freeway Work Zones

The 1985 HCM has added a most useful section giving detailed information concerning the capacity impacts of a variety of freeway maintenance and construction activities. With the increasing pace of freeway rehabilitation, the impacts of maintenance and construction operations must be better understood, and can now be planned for in a more systematic way.

Table 6-1 shows general criteria for the capacity of freeway lanes under various lane-closure situations.

**TABLE 6-1**

**AVERAGE CAPACITIES OF FREEWAY WORK ZONES**

| NUMBER OF LANES A NORMAL | NUMBER OF LANES B OPEN | NUMBER OF STUDIES | AVERAGE CAPACITY (VPH) | AVERAGE CAPACITY (VPHPL) |
|---|---|---|---|---|
| 3 | 1 | 7 | 1,170 | 1,170 |
| 2 | 1 | 8 | 1,340 | 1,340 |
| 5 | 2 | 8 | 2,740 | 1,370 |
| 4 | 2 | 4 | 2,960 | 1,480 |
| 3 | 2 | 9 | 2,980 | 1,490 |
| 4 | 3 | 4 | 4,560 | 1,520 |

The manual gives greater detail on impacts of different types of maintenance and construction activities, and documents the variability of capacity impacts. Further applications of capacity vs. arrival flows are used to estimate aggregate delays and queue build-up as a result of maintenance and constructions activities.

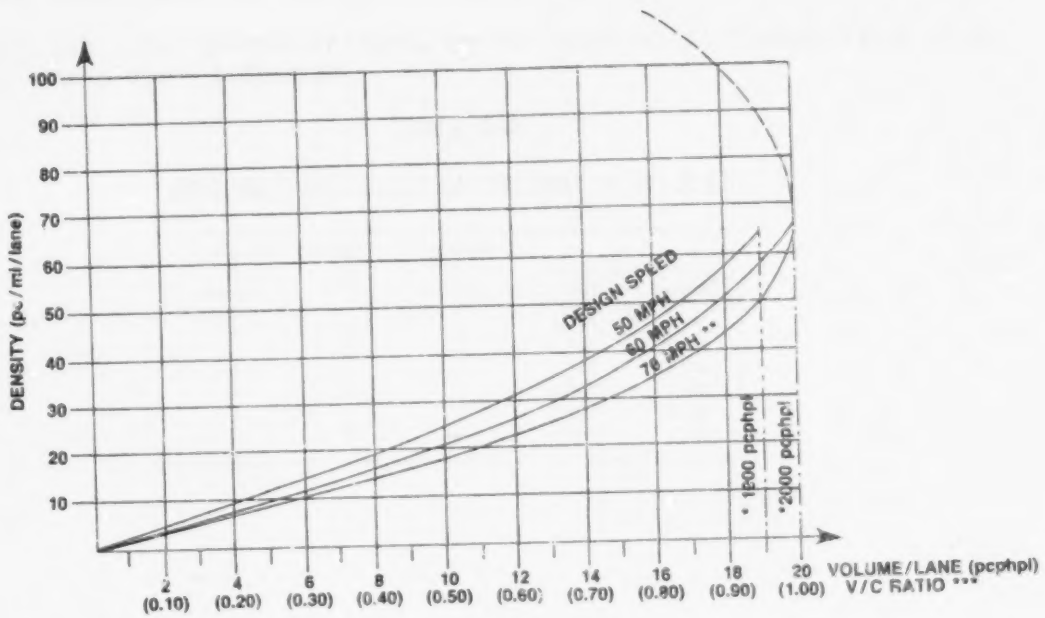
## Impact of High-Occupancy Vehicle Lanes on Freeway Operations

High-occupancy vehicle lanes are becoming a more frequently-used alternative for increasing the person-capacity of freeways without major construction. The 1985 HCM illustrates how standard analysis procedures can be applied to estimate the impact of such lanes on the performance on the remainder of the freeway. Chapter 12 of the HCM, "Transit," deals in greater depth with the planning and operation of high-occupancy vehicle lanes themselves.

## Chapter 7: Multilane Highways

Chapter 7 of the 1985 HCM treats the capacity analysis of uniterrupted flow sections of multilane highways. A guideline of 2-mile minimum spacing between signals is given for uninterrupted flow to exist. For closer spacings, the facility would be classified as an arterial, and would be analyzed using procedures of Chapter 11.

Procedures for multilane highways parallel those for basic freeway sections, although many of the values and relationships used differ. Figures 7-1 and 7-2 depict the basic relationships between speed and flow and density and flow for multilane highways under ideal conditions. Speed is more sensitive to flow on multilane highways than on freeways due to the higher levels of side- and median-frictions that exist on surface facilities. These frictions involve vehicles entering and leaving the roadway to accessdriveways and unsignalized intersections. Many such movements involve left turns, which are a serious frictional element on multilane highways.



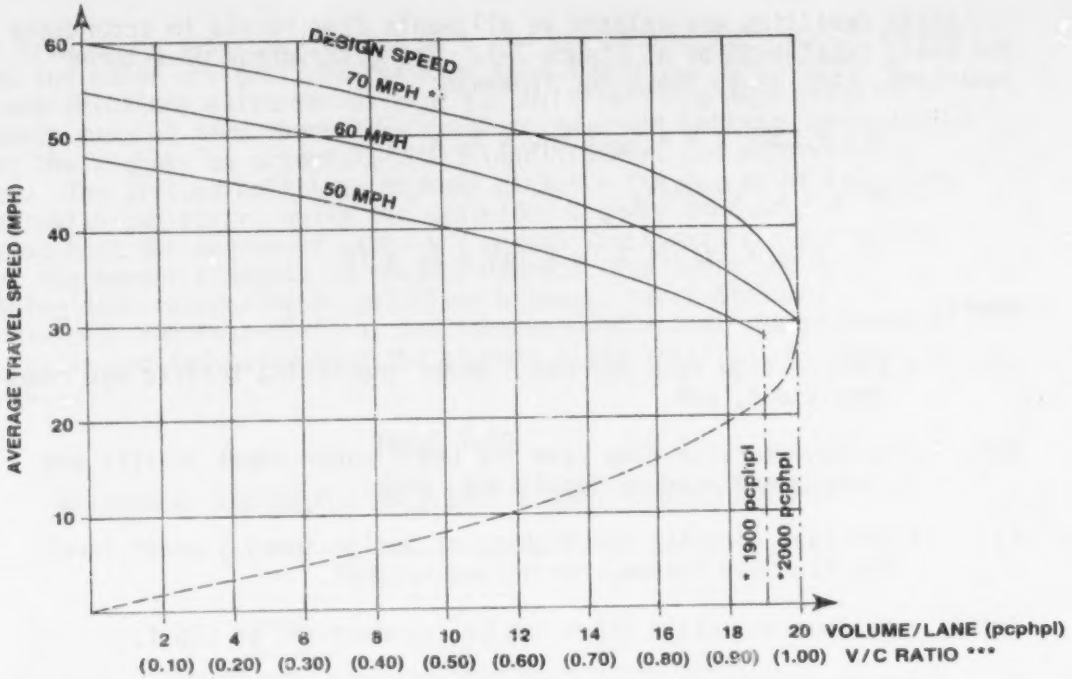

* capacity

** reflects 55 MPH speed limit

*** v/c ratio based on capacity of 2000 pcphpl, applies only to 60 and 70 MPH design speeds

FIGURE 7-1

DENSITY-FLOW CHARACTERISTICS FOR
UNINTERRUPTED FLOW SEGMENTS OF MULTILANE HIGHWAYS




* capacity

** reflects 55 MPH speed limit

*** v/c ratio based on capacity of 2000 pcphpl, applies only to 60 and 70 MPH design speeds

FIGURE 7-2

SPEED-FLOW CHARACTERISTICS FOR

UNINTERRUPTED FLOW SEGMENTS OF MULTILANE HIGHWAYS

Levels of service for multilane highways are defined by density ranges which are the same as those for freeways:

| | |
|---|---|
| LOS A | $\leq$ 12 pc/mi/ln |
| LOS B | $\leq$ 20 pc/mi/ln |
| LOS C | $\leq$ 30 pc/mi/ln |
| LOS D | $\leq$ 42 pc/mi/ln |
| LOS E | $\leq$ 67 pc/mi/ln |
| LOS F | $>$ 67 pc/mi/ln |

These densities are related to allowable flow levels in accordance with the basic relationships of Figure 7-1. The methodology uses three equations, similar to those for freeways:

$$SF_i = MSF_i \times N \times f_w \times f_{HV} \times f_p \times f_E$$

$$MSF_i = c_j \times (v/c)_i$$

$$SF_i = c_j \times (v/c)_i \times f_w \times f_{HV} \times f_p \times f_E$$

where:

$SF_i$ = service flow rate for LOS i under prevailing traffic and roadway conditions, vph.

$MSF_i$ = maximum service flow rate for LOS i under ideal traffic and roadway and roadway conditions, pcph.

$c_j$ = per lane capacity for highway of design speed j under ideal traffic and roadway conditions, pcphpl.

$(v/c)_i$ = maximum v/c ratio which can be accommodated at LOS i.

N = number of lanes in one direction.

$f_w$ = adjustment factor for narrow lanes and restricted lateral clearance.

$f_{HV}$ = adjustment factor for heavy vehicles.

$f_p$ = adjustment factor for driver population.

$f_E$ = adjustment factor for multilane highway classification and environment.

In operational analysis, the equation is solved for v/c ratio, with the service flow rate, SF, set equal to the existing or projected rate of flow. In design analysis, the equation is solved for N, setting SF equal to the directional design hour flow rate.

Most of the tabulations of Chapter 7 are similar to those in Chapter 3 for basic freeway sections. A table of v/c ratios and maximum service flow rates, MSF, is provided for ideal conditions. Adjustments for lane width and lateral clearance, $f_w$, differ from those for freeways, but are presented and used in a similar format.

The heavy vehicle adjustment factor, $f_{HV}$, and the driver population factor, $f_p$, are the same for multilane highways as for basic freeway sections, although all tables are repeated in Chapter 7 for convenience of the user.

The multilane highway classification and environment factor, $f_E$, is uniquely applied to multilane highways. Remember that the most unique factor in the operation of multilane surface facilities is the degree of

friction created by vehicles turning onto and off of the facility to access driveways and other unsignalized roadside access points. The degree to which these frictions exist varies widely. Multilane highways exist in environments ranging from thoroughly rural to suburban settings which border on making the highway an arterial. Multilane highways can be divided or undivided. The divided multilane highway controls the degree of left-turn frictions which can occur, while the undivided highway does not. Recognizing that the degree of side- and median-frictions is a continuous variable, the manual attempts to roughly address this variation by establishing four categories of multilane highway: rural-divided; rural-undivided; suburban-divided; and suburban-undivided. An adjustment factor, $f_E$, is applied based upon the highway classification, as shown in Table 7-10.

**TABLE 7-10**

**ADJUSTMENT FACTOR FOR MULTILANE HIGHWAY CLASSIFICATION**

| TYPE | DIVIDED | UNDIVIDED |
|---|---|---|
| Rural | 1.00 | 0.95 |
| Suburban | 0.90 | 0.80 |

Chapter 7 of the 1985 HCM will be the first chapter to undergo complete revision. A National Cooperative Highway Research Program project is currently underway to conduct widespread data collection and analysis, hopefully leading to an improved methodology, particularly in the area of quantifying and evaluating varying degrees of side- and median-frictions. The new chapter is expected in 1988.

## CHAPTER 8: TWO-LANE HIGHWAYS

The 1985 HCM introduces an entirely new procedure for the analysis of rural two-lane highways. In analyzing 2-lane highways, it is important to keep in mind two critical points:

1. Two-lane highway operation is unique, and is significantly different from other forms of uninterrupted flow. The reason for this uniqueness is the interaction between directional flows, i.e., the characteristic that flow in one direction influences flow in the other.

2. The most significant operational problem on two-lane highways is the formation of platoons behind slow-moving vehicles as passing opportunities are restricted either by opposing flow, terrain, or both. A new LOS measure, percent time delay, is introduced to describe this characteristic.

As passing must be done in the opposing lane, traffic in one direction both limits and affects flow in the other. Thus, like the 1965 HCM, the 1985 HCM treats two-lane highway capacity as a total analysis of flow in both directions. The methodology in the 1985 HCM is, however, new -- both in concept and in computational procedures.

Few two-lane highways are ever observed operating at or near capacity. This is because most two-lane highways will experience severe operating problems at flow levels well below capacity. These often require improvements creating three- or four-lane alignments. Based upon simulation results, and observations of two-lane highway flows on short sections, such as two-lane bridges and tunnels fed by multilane approaches, the capacity of two-lane highways has been established as a total of 2,800 pcph in both directions under ideal conditions. This is significantly higher than the 2,000 pcph stated in the 1965 HCM, and reflects a better understanding of the dynamics of two-lane highway flow. Ideal conditions, however, cover a wide range of variables, including: 60-mph design speed, 12-ft lanes, 6-ft shoulders, all passenger cars in the traffic stream, level terrain, no "NO PASSING" zones, and a 50/50 directional distribution of traffic.

### Level of Service Criteria

The 1985 HCM introduces a new LOS measure for two-lane highways: percent time delay. Percent time delay is defined as the percent of total travel time spent by drivers in platoons behind slow-moving vehicles, unable to pass. Figure 8-1b depicts the relationship between percent time delay and total two-way volume for two-lane highways.

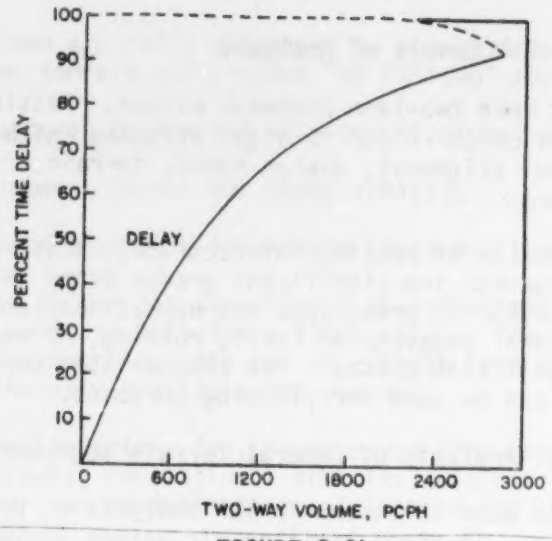



**FIGURE 8-1b**

**PERCENT TIME DELAY VS. VOLUME FOR TWO-LANE HIGHWAYS**

Note that even at a volume of 1,200 pcph, which is less than half the capacity of a two-lane facility, that percent time delay is over 60 percent. Thus, two-lane highways operating at small fractions of their capacity experience severe operating problems. Percent time delay rises rapidly with volume for the following reason. As volume increases, the demand for passing maneuvers increases, while at the same time, capacity for passing maneuvers decreases. Note also that Figure 8-1b is for ideal conditions; non-ideal conditions will make percent time delay even worse.

Percent time delay is a difficult parameter to measure. The manual suggests a surrogate: the percentage of vehicles traveling at headways of less than 5 seconds.

The following criteria are used to define levels of service on two-lane highways:

| | |
|---|---|
| LOS A | $\leq$ 30% time delay |
| LOS B | $\leq$ 45% time delay |
| LOS C | $\leq$ 60% time delay |
| LOS D | $\leq$ 75% time delay |
| LOS E | $\leq$100% time delay |
| LOS F | 100% time delay |

Note from Figure 8-1b that under ideal conditions, LOS E or 75% time delay is reached at a volume of approximately 1,700 pcph. Thus, for ideal conditions, levels of service A-D cover a range of 1,700 pcph while LOS E alone covers a range of 1,100 pcph. This is merely a reflection of the reality that severe operating difficulties arise on two-lane highways at relatively low volumes.

## Levels of Analysis

Passing interactions make two-lane highways unique. Passing is affected by many variables, all of which relate to sight distance which is restricted by horizontal and vertical alignment, design speed, terrain, roadside objects, and other factors.

Because of the complexity of passing interactions, it is not possible to treat general terrain segments and significant grades using the same methodology. Thus, the 1985 HCM prescribes two operational analysis procedures: one for general segments of level, rolling, or mountainous terrain; another for significant grades. The chapter also contains a table of AADT's vs. LOS which can be used for planning purposes.

## Operational Analysis of General Terrain Segments

Table 8-1 depicts the base relationship for analysis of general terrain two-lane highway segments. It gives limiting v/c values for each LOS as affected by terrain and percent "NO PASSING" zones in the section.

**TABLE 8-1**

**LEVEL OF SERVICE CRITERIA FOR TWO-LANE HIGHWAYS**
**GENERAL TERRAIN SEGMENTS**

| | | v/c RATIO[a] | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | LEVEL TERRAIN | | | | | | | ROLLING TERRAIN | | | | | | | MOUNTAINOUS TERRAIN | | | | | | | |
| | PERCENT TIME DELAY | AVG[b] SPEED | PERCENT NO PASSING ZONES | | | | | | AVG[b] SPEED | PERCENT NO PASSING ZONES | | | | | | AVG[b] SPEED | PERCENT NO PASSING ZONES | | | | | |
| LOS | | | 0 | 20 | 40 | 60 | 80 | 100 | | 0 | 20 | 40 | 60 | 80 | 100 | | 0 | 20 | 40 | 60 | 80 | 100 |
| A | ≤ 30 | ≥ 58 | 0.15 | 0.12 | 0.09 | 0.07 | 0.05 | 0.04 | ≥ 57 | 0.15 | 0.10 | 0.07 | 0.05 | 0.04 | 0.03 | ≥ 56 | 0.14 | 0.09 | 0.07 | 0.04 | 0.02 | 0.01 |
| B | ≤ 43 | ≥ 55 | 0.27 | 0.24 | 0.21 | 0.19 | 0.17 | 0.16 | ≥ 54 | 0.26 | 0.23 | 0.19 | 0.17 | 0.15 | 0.13 | ≥ 54 | 0.25 | 0.20 | 0.16 | 0.13 | 0.12 | 0.10 |
| C | ≤ 60 | ≥ 52 | 0.43 | 0.39 | 0.36 | 0.34 | 0.33 | 0.32 | ≥ 51 | 0.42 | 0.39 | 0.35 | 0.32 | 0.30 | 0.28 | ≥ 49 | 0.39 | 0.33 | 0.28 | 0.23 | 0.20 | 0.16 |
| D | ≤ 75 | ≥ 50 | 0.64 | 0.62 | 0.60 | 0.59 | 0.58 | 0.57 | ≥ 49 | 0.62 | 0.57 | 0.52 | 0.48 | 0.46 | 0.43 | ≥ 45 | 0.58 | 0.50 | 0.45 | 0.40 | 0.37 | 0.33 |
| E | > 75 | ≥ 45 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | ≥ 40 | 0.97 | 0.94 | 0.92 | 0.91 | 0.90 | 0.90 | ≥ 35 | 0.91 | 0.87 | 0.84 | 0.82 | 0.80 | 0.78 |
| F | 100 | < 45 | — | — | — | — | — | — | < 40 | — | — | — | — | — | — | < 35 | — | — | — | — | — | — |

[a] Ratio of flow rate to an ideal capacity of 2,800 pcph in both directions.
[b] Average travel speed of all vehicles (in mph) for highways with design speed ≥ 60 mph; for highways with lower design speeds, reduce speed by 4 mph for each 10-mph reduction in design speed below 60 mph; assumes that speed is not restricted to lower values by regulation.

This table is unique, in that it is not strictly for ideal conditions. It essentially incorporates adjustments for terrain and percent "NO PASSING" zones. Because of the passing interactions, capacity of two-lane highways is sensitive to terrain -- the only type of facility where this is so. Thus, not all v/c ratios for LOS E -- capacity -- are 1.00. All v/c ratios are based upon the ideal capacity of 2,800 pcph.

The general relationship for analysis of general terrain segments is as follows:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_{HV} \times f_w$$

where:

$SF_i$ = total service flow rate in both directions under prevailing roadway and traffic conditions for LOS i.

$(v/c)_i$ = maximum v/c ratio which can be accommodated at LOS i for a given terrain and percent "NO PASSING" zones.

$f_d$ = adjustment factor for directional distribution of traffic.

$f_{HV}$ = adjustment factor for heavy vehicles.

$f_w$ = adjustment factor for narrow lanes and shoulders.

The adjustment factor for directional distribution, $f_d$, varies from 1.00 for a 50/50 split to 0.71 for a 100/0 split. The latter corresponds to 2,000 pcph, the maximum flow rate for any single lane where there is a second lane continuously available for passing.

The heavy vehicle factor, $f_{HV}$, is computed from passenger car equivalents for trucks, recreational vehicles, and buses, as in previous methods. Passenger car equivalents are tabulated for level, rolling, and mountainous terrain.

The lane and shoulder width factor, $f_w$, is tabulated in a manner similar to other chapters. For multilane highways and freeways, however, the concern was for lateral clearance, i.e., the distance from the edge of the travel lane to the nearest roadside objects. For two-lane highway analysis, the concern is for the width of usable shoulder, which can allow slow-moving vehicles to pull aside to let faster vehicles pass.

Another unique feature of the two-lane highway methodology is that both the heavy vehicle and lane width adjustments vary with level of service. This could make solution for the level of service an interactive procedure. As an alternative to this, the chapter provides worksheets on which five values of service flow rate, SF, can be computed -- one for each LOS A-E. These values are then compared to the actual or projected flow rates to determine LOS.

### Operational Analysis of Grades of Significant Length

Any sustained grade of more than 3% and longer than 1/2 mile, or 3% or less and longer than one mile is analyzed as a specific grade of significant length. The dynamics of flow on a sustained two-lane highway grade are substantially different from those on general terrain segments. Formation of platoons on the upgrade becomes more common, while passing becomes more difficult. Average travel speeds on upgrades vary widely, depending upon the length and severity of grade, mix of traffic, passing opportunities, and other factors.

Level of service criteria are keyed to the average upgrade travel speed, which is most sensitive to grade. Levels are established to relate to approximately the same percent time delay values as on general terrain sections. Criteria are as follows:

| | |
|---|---|
| LOS A | $\geq$ 55 mph |
| LOS B | $\geq$ 50 mph |

| | |
|---|---|
| LOS C | $\geq$ 45 mph |
| LOS D | $\geq$ 40 mph |
| LOS E | $\geq$ 25-40 mph |

These criteria highlight a significant complication in the analysis of two-lane sustained grades: the operating conditions at capacity are not constant. Depending upon the length and severity of grade, as well as other traffic and roadway conditions, capacity operations can occur at average upgrade speeds anywhere in the 25-40 mph range. Thus, part of the analysis of two-lane highway grades is the determination of capacity and the operating conditions under which it occurs.

The general relationship for sustained grades on two-lane highways is as follows:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_g \times f_{HV}$$

where:

$SF_i$ = total two-way service flow rate for LOS i under prevailing roadway and traffic conditions, vph.

$(v/c)_i$ = maximum v/c ratio for LOS i for the given percent grade and percent "NO PASSING" zones.

$f_d$ = adjustment factor for directional distribution of traffic.

$f_w$ = adjustment factor for narrow lanes and shoulders.

$f_g$ = adjustment factor for grades.

$f_{HV}$ = adjustment factor for heavy vehicles.

Table 8-7 illustrates the basic relationship governing values of v/c. It has several interesting characteristics. Note that values are not specifically related to LOS, but to values of average upgrade travel speed. For LOS A-D, there is a one-to-one relationship between speed and LOS, but for LOS E, the average upgrade travel speed is unknown. Use of speeds as a key, rather than LOS, allows a relationship between speed and SF to be found and plotted which is later used to determine capacity and speed and capacity.

## TABLE 8-7

## VALUES OF v/c RATIO FOR TWO-LANE HIGHWAY GRADES

| PERCENT GRADE | AVERAGE UPGRADE SPEED (MPH) | PERCENT NO PASSING ZONES | | | | | |
|---|---|---|---|---|---|---|---|
| | | 0 | 20 | 40 | 60 | 80 | 100 |
| 3 | 55 | 0.27 | 0.23 | 0.19 | 0.17 | 0.14 | 0.12 |
| | 52.5 | 0.42 | 0.38 | 0.33 | 0.31 | 0.29 | 0.27 |
| | 50 | 0.64 | 0.59 | 0.55 | 0.52 | 0.49 | 0.47 |
| | 45 | 1.00 | 0.95 | 0.91 | 0.88 | 0.86 | 0.84 |
| | 42.5 | 1.00 | 0.98 | 0.97 | 0.96 | 0.95 | 0.94 |
| | 40 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| 4 | 55 | 0.25 | 0.21 | 0.18 | 0.16 | 0.13 | 0.11 |
| | 52.5 | 0.40 | 0.36 | 0.31 | 0.29 | 0.27 | 0.25 |
| | 50 | 0.61 | 0.56 | 0.52 | 0.49 | 0.47 | 0.45 |
| | 45 | 0.97 | 0.92 | 0.88 | 0.85 | 0.83 | 0.81 |
| | 42.5 | 0.99 | 0.96 | 0.95 | 0.94 | 0.93 | 0.92 |
| | 40 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| 5 | 55 | 0.21 | 0.17 | 0.14 | 0.12 | 0.10 | 0.08 |
| | 52.5 | 0.36 | 0.31 | 0.27 | 0.24 | 0.22 | 0.20 |
| | 50 | 0.57 | 0.49 | 0.45 | 0.41 | 0.39 | 0.37 |
| | 45 | 0.93 | 0.84 | 0.79 | 0.75 | 0.72 | 0.70 |
| | 42.5 | 0.97 | 0.90 | 0.87 | 0.85 | 0.83 | 0.82 |
| | 40 | 0.98 | 0.96 | 0.95 | 0.94 | 0.93 | 0.92 |
| | 35 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| 6 | 55 | 0.12 | 0.10 | 0.08 | 0.06 | 0.05 | 0.04 |
| | 52.5 | 0.27 | 0.22 | 0.18 | 0.16 | 0.14 | 0.13 |
| | 50 | 0.48 | 0.40 | 0.35 | 0.31 | 0.28 | 0.26 |
| | 45 | 0.49 | 0.76 | 0.68 | 0.63 | 0.59 | 0.55 |
| | 42.5 | 0.93 | 0.84 | 0.78 | 0.74 | 0.70 | 0.67 |
| | 40 | 0.97 | 0.91 | 0.87 | 0.83 | 0.81 | 0.78 |
| | 35 | 1.00 | 0.96 | 0.95 | 0.93 | 0.91 | 0.90 |
| | 30 | 1.00 | 0.99 | 0.99 | 0.98 | 0.98 | 0.98 |
| 7 | 55 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | 52.5 | 0.13 | 0.10 | 0.08 | 0.07 | 0.05 | 0.04 |
| | 50 | 0.34 | 0.27 | 0.22 | 0.18 | 0.15 | 0.12 |
| | 45 | 0.77 | 0.65 | 0.55 | 0.46 | 0.40 | 0.35 |
| | 42.5 | 0.86 | 0.75 | 0.67 | 0.60 | 0.54 | 0.48 |
| | 40 | 0.93 | 0.82 | 0.75 | 0.69 | 0.64 | 0.59 |
| | 35 | 1.00 | 0.91 | 0.87 | 0.82 | 0.79 | 0.76 |
| | 30 | 1.00 | 0.95 | 0.92 | 0.90 | 0.88 | 0.86 |

[a] Ratio of flow rate to ideal capacity of 2,800 pcph, assuming passenger-car operation is unaffected by grade.
NOTE: Interpolate for intermediate values of "Percent No Passing Zone"; round "Percent Grade" to the next higher integer value.

Values of the directional distribution factor, $f_d$, are tabulated, and differ from those for general terrain sections. On sustained grades, a 30/70 split differs from a 70/30 split, depending upon which percentage is on the upgrade. On general terrain segments, it does not matter which direction is which.

The lane and shoulder width adjustment factor, $f_w$, is the same as for general terrain segments.

The grade factor, $f_g$, and the heavy vehicle factor, $f_{HV}$, are the result of a complex algebraic formulation which addresses two issues. Trucks and other heavy vehicles occupy more space than passenger cars on grades, and have a particularly deleterious effect on platoon formation on grades. Passenger cars, however, also behave differently on two-lane grades than on general terrain segments. The two adjustments address both of these issues in an interwoven manner for computational ease.

A worksheet is provided to allow computation of a service flow rate, SF, for various values of average upgrade travel speed. For LOS A-D, these will correlate directly to service flow rates for each LOS. LOS E cannot be identified, as the speed at which it occurs is unknown. A second relationship has been calibrated which relates capacity to the critical speed at which it occurs:

$$S_c = 25 + 3.75(v_c/1000)^2$$

where:

$S_c$ = critical speed at which capacity occurs, mph.

$V_c$ = flow rate at capacity, vph.

Capacity and the speed at which it occurs is defined by the intersection of this curve with a curve plotted for SF vs. average upgrade speed, using the values computed as previously described. A worksheet is provided for this purpose as well, and is illustrated in Figure 8-9b, which shows a sample solution for capacity using the graphic procedure.

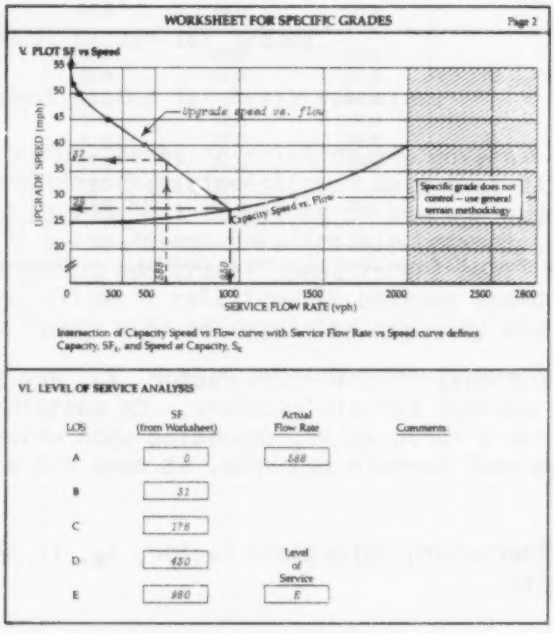



**FIGURE 8-9b**

**SAMPLE SOLUTION FOR CAPACITY AND CRITICAL SPEED ON A TWO-LANE GRADE**

## System Planning Criteria

The 1985 HCM contains a tabulation of approximate AADT's which can be served by two-lane highways without violating stated levels of service during peak periods. These are given for a set of standard conditions, and are only valid for general terrain sections. They provide an easy look-up for general planning purposes, and are useful for identifying sections likely to experience difficulties in the future.

## Improving Two-Lane Highways

Operational problems on two-lane highways can be alleviated by a variety of measures short of reconstruction as multilane segments. The 1985 HCM contains a section which discusses such improvement possibilities as:

- climbing lanes
- periodic passing lanes/alternating passing lanes
- turnouts
- continuous left-turn lanes
- intersection treatments
- use of paved shoulders
- short 4-lane sections

The manual treats most of these in a qualitative manner, as specific information on the impact of each on capacity and operations is not available.

## CHAPTER 9: SIGNALIZED INTERSECTIONS

The most dramatic change in the 1985 HCM is in procedures for signalized intersections, which differ substantially from those of the 1965 HCM and go considerably beyond those published in Circular 212 in 1980. Signalized intersections are the most complex points in any traffic system, and procedures reflect this complexity. The 1985 HCM treats a broader range of variables in greater depth than any previous analysis technique for signalized intersections.

### Basic Approach

The methodology adopted for the 1985 HCM is based upon Critical Movement Analysis (CMA). Critical movement analysis focuses on the identification and analysis of those movements in particular lanes or groups of lanes which control the requirements for green time allocation. Once critical movements are identified and provided for, all other movements will operate as well or better.

It is important to note that capacity analysis computations for signalized intersections are reciprocal to signal timing computations. In the former case, the signal timing is known, and the LOS for known traffic and roadway conditions is computed. In the latter, known traffic and roadway conditions are manipulated to find a signal timing which will provide for a given LOS. If the same methodology is not used for both types of computations, the results of signal timing and capacity analyses will be inconsistent and difficult to explain. Chapter 9 of the 1985 HCM is structured to allow both types of computations, and a major Appendix is included on signal timing.

### Levels of Service

From the moment the 1965 HCM was published, criticism was directed at the use of Load Factor as a measure for level of service. It was difficult to measure, and proved not to be well correlated to delay, as was originally assumed. Numerous papers by noted researchers pressed the argument that delay was the only reasonable measure for intersection LOS. Very early in the process of developing the 1985 HCM, delay was adopted as the measure for intersection LOS.

As the analysis procedures developed, it became clear that use of delay, while theoretically satisfying, led to considerable computational difficulties and to basic changes in the meaning and interpretation of level of service. The 1985 HCM defines level of service for signalized intersections as follows:

| | |
|---|---|
| LOS A | $\leq$ 5.0 seconds |
| LOS B | $\leq$ 15.0 seconds |
| LOS C | $\leq$ 25.0 seconds |
| LOS D | $\leq$ 40.0 seconds |
| LOS E | $\leq$ 60.0 seconds |
| LOS F | $>$ 60.0 seconds |

These criteria refer to average stopped delay per vehicle during a 15-minute analysis period.

Delay is a very complex measure, which depends upon a number of factors. Research has indicated that delay depends primarily on the following four factors, listed in order of decreasing importance:

1) quality of signal progression

2) cycle length

3) green times

4) v/c ratios

The order of the listing presents the problem. Delay is not highly dependent on v/c ratio. Low delay can be accomplished at an intersection with a high v/c ratio if signals are favorably progressed, and the timing is efficient. Conversely, high delay can occur at an intersection with a low v/c ratio where progression is poor and timing is inefficient. Very low v/c ratios, for example, are indicative of excess green time in the cycle. Such excess green times may lead to high delays. On the other hand, v/c ratios approaching capacity can also lead to high delays as individual cycles fail more often. The following point is critical: it is not necessary to achieve low v/c ratios to have a good level of service based upon delay.

Another vexing result occurs with delay-based levels of service: LOS F no longer automatically indicates oversaturation. It merely depicts delay over 60 seconds per vehicle. This level of delay can and does occur at v/c ratios as low as 0.80-0.85, where signal progression and timing are inefficient. Thus, for intersections, level of service F DOES NOT always signify a breakdown or a case in which demand exceeds capacity.

Because of these characteristics, the 1985 HCM emphasizes that capacity and level of service analyses of signalized intersections are somewhat independent of one another. Analysis of both v/c ratios and delay levels is required to get a full picture of intersection operations.

## Operational Analysis

Operational analysis of signalized intersections is complex and time-consuming if all computations are done by hand. Six worksheets are provided, and at least five are needed to complete an analysis. The procedure evaluates a broad range of factors affecting operations, falling generally into three categories: geometrics, signal control, and traffic.

Geometric factors include the number and use of lanes, lane widths, grades, location of bus stops, and similar factors. Control variables involve the type, phasing, and timing of the traffic signal. Traffic factors include the usual composition variables, parking activity, pedestrian crossing flows, and vehicular flows by movement.

Because of the complexity of the analysis procedures, computations are done in a modular fashion, as depicted in Figure 9-1.

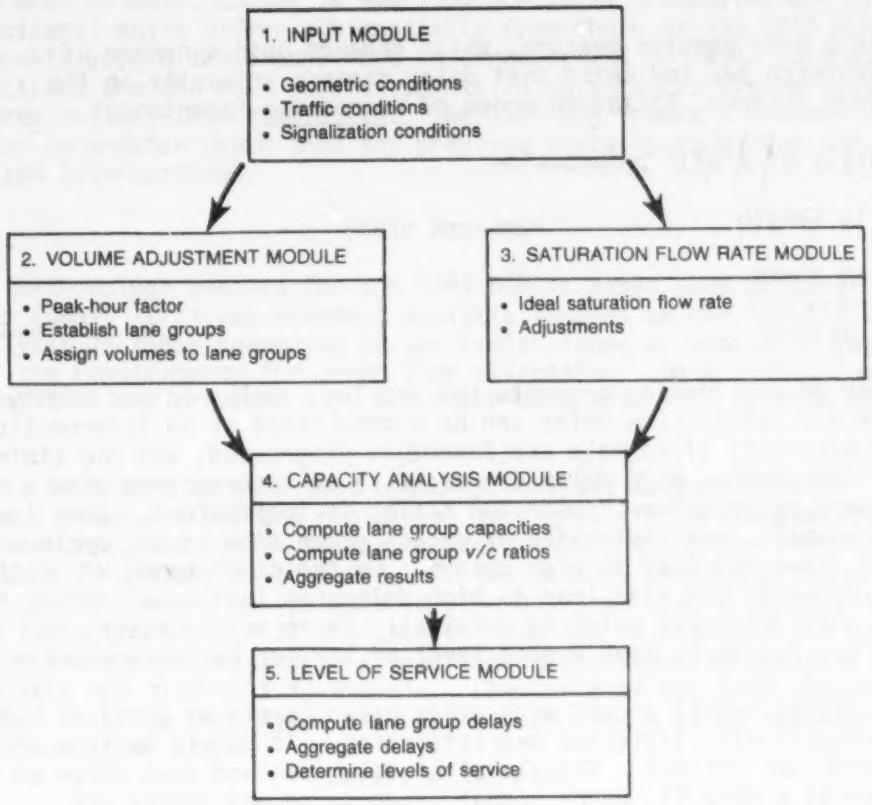



FIGURE 9-1

MODULAR STRUCTURE OF OPERATIONAL ANALYSIS
OF SIGNALIZED INTERSECTIONS

The input module is simply a worksheet which allows all of the variables needed for analysis to be easily summarized. The 1985 HCM requires more detailed data than previous analysis techniques. Some of the additional information now required includes:

1. Pedestrian flows in all crosswalks.

2. Number of parking movements per hour into and out of parking spaces within 250 ft of the intersection.

3. Arrival type of each approach. Five arrival types are defined and are used to evaluate the quality of progression. Dense platoons arriving at the beginning of the RED phase is the worst case; dense platoons arriving at the beginning of the GREEN phase is the best case.

4. Percent grade on all approaches.

Because of the large number of input variables needed to conduct an operational analysis, the manual provides a tabulation of default values for many of these which may not be readily available from data. Key values, however, such as the arrival type, must be evaluated in the field or projected.

Two major determinations are completed in the volume adjustment module. All hourly volumes are converted to peak flow rates by dividing by the appropriate peak hour factors. Most importantly, the intersection is divided into lane groups for analysis. In general, all exclusive turning lanes are considered as separate lane groups. Approaches containing only shared lanes are considered as single lane groups.

The saturation flow module computes the saturation flow rate for each lane group under prevailing roadway, traffic, and control conditions. The following equation is used:

$$s = s_o \, N \, f_w \, f_{HV} \, f_g \, f_p \, f_{bb} \, f_a \, f_{RT} \, f_{LT}$$

where:

$s$ = saturation flow rate for a lane group under prevailing traffic, roadway, and control conditions, vphg.

$s_o$ = saturation flow rate per lane for ideal conditions; 1,800 pcphgpl unless locally calibrated value is available.

$N$ = number of lanes in the lane group.

$f_w$ = adjustment factor for lane width.

$f_{HV}$ = adjustment factor for heavy vehicles.

$f_g$ = adjustment factor for grade.

$f_p$ = adjustment factor for parking activity.

$f_{bb}$ = adjustment factor for bus blockage.

$f_a$ = adjustment factor for area type.

$f_{RT}$ = adjustment factor for right turns.

$f_{LT}$ = adjustment factor for left turns.

Most of these adjustment factors are simple tabular look-ups. Right- and left-turn factors are more complex. Turns can occur under eight differing combinations of geometrics and signalization. Turns can be made from exclusive lanes or from mixed traffic lanes. Signalization can provide for protected turns, permitted turns, or a combination of protected plus permitted turns. Protected turns are made with no conflicting vehicular or pedestrian flow. Permitted turns are made through a conflicting vehicular or pedestrian flow.

Computation of the left-turn adjustment factor for cases involving permitted left-turn phasing is complex. Permitted left turns involve a complicated equilibrium in which left-turners seek gaps in the opposing traffic stream. At the end of the phase, several left-turning vehicles may proceed through the intersections on the change interval. A special worksheet is provided for computing the left-turn factor for any case involving permitted turns.

In the capacity analysis module, capacities and v/c ratios for each lane group are computed, as is a critical v/c ratio for the intersection as a whole. Computations use the results of the input, volume adjustment, and saturation flow modules.

The capacity analysis module results in v/c ratios for all lane groups, given that the signal timing is known. The manual gives procedures for determining signal timing if it is not specified as an input to the analysis. Signal timing computations require that the type of signal control (pretimed, semiactuated, actuated) and phase plan be specified. Cycle length and green times may then be estimated and used to complete the capacity analysis.

Once v/c ratios are known, the level of service module estimates the average stopped delay per vehicle in each lane group, and aggregates these estimates for approaches and for the intersection as a whole. Delay is computed for each lane group as follows:

$$d_1 = \frac{0.38\ C\ (1 - g/C)^2}{[1 - (g/C)(X)]}$$

$$d_2 = 173\ X_2\ [(X-1) + \sqrt{(X-1)^2 + 16X/c}\ ]$$

$$d = [d_1 + d_2] \times PF$$

where:

$d_1$ = uniform delay from Webster's Delay Equation, secs/veh.

$d_2$ = incremental delay due to random arrivals and individual cycle failures, secs/veh.

$d$ = total stopped delay per vehicle, secs/veh.

$PF$ = progression factor, based upon arrival type.

$C$ = cycle length, secs.

$g$ = green time for lane group, secs.

$X$ = v/c ratio for lane group.

$c$ = capacity of lane group, vph.

Delay values for individual lane groups are aggregated to evaluate approaches and the overall intersection by taking weighted averages based upon the relative flows in each lane group. Levels of service are assigned based upon delay estimates.

Six worksheets are provided to assist in operational analysis computations -- one for each module, and a supplemental worksheet for left-turn factor computations. Regular users of the procedure will program spreadsheets or programmable calculators to do many of the computations, and the use of computer programs will be of great assistance.

## Planning Analysis

The 1985 HCM does provide a planning analysis technique for intersections which is considerably less complex than the operational analysis procedure. There are no adjustment factors, and details of signal timing need not be known. Because of the simplicity of the procedure, however, it yields only a gross estimate of whether or not demand will exceed capacity. It does not yield delay or level of service evaluations, and should only be applied to general analyses of future intersections.

Chapter 9 contains a great deal of detailed information including several appendices. Appendices address geometric design, signalization, and field measurement of delay and saturation flow rates. The chapter should be carefully studied before use, as both computations and interpretations of results are complex.

## CHAPTER 10: UNSIGNALIZED INTERSECTIONS

Chapter 10 is another new addition to the 1985 HCM. A methodology is provided to analyze the capacity and operations at two-way STOP- or YIELD-controlled approaches. Information on the capacity of four-way STOP-controlled intersections is also provided.

The basic methodology was developed in Germany in the early 1970's. The procedure has not been extensively validated in the United States.

The method is based upon a strict priority order of gap utilization at STOP-or YIELD-controlled approaches:

1. Right turns from the minor street.
2. Left turns from the major street.
3. Through movements from the minor street.
4. Left turns from the minor street.

Right turns from the minor street merge into gaps in the right-hand lane of the major street. The movement is relatively simple, and does not impede other vehicles seeking to use the same gaps.

Left turns from the major street seek gaps in the opposing major street flow. While waiting for such a gap, they are generally positioned in the middle of the intersection, preventing lower priority movements from using any available gaps until their maneuver is completed.

Through vehicles from the minor street must cross through the entire major street flow. The through vehicle impedes an opposing left-turning vehicle from the minor street, which follows a conflicting path.

The left turning vehicle from the minor street faces the most complex movement. Gaps must be sought in the entire major street flow, and conflicts with opposing through vehicles must be avoided.

The priority of gap utilization is important. Large volumes of left-turning vehicles from the major street, for example, could use so many of the available gaps that lower priority movements become congested, facing a capacity deficiency.

Computations are done in order of movement priority. Thus, right turns from the minor street are analyzed first, followed by major street left turns, minor street through movements, and minor street left turns. This allows an evaluation of the impact of higher-priority movements on lower priority movements.

Capacities for each movement are estimated in three steps:

1. Potential capacity is estimated. Potential capacity assumes that each movement has use of an exclusive lane, and is able to use all available gaps in the major traffic stream.

2. Movement capacity is estimated. Movement capacity accounts for the impact of impedance of high-priority movements on low priority movements.

3. Shared-Lane capacity is estimated. Shared-lane capacity accounts for the sharing of lanes by various minor street movements.

Level of service is related to "reserve capacity," $c_r$, which is computed as the shared-lane capacity minus the lane volume. Levels of service are related to reserve capacity as shown in Table 10-3.

**TABLE 10-3**

**LEVELS OF SERVICE CRITERIA FOR UNSIGNALIZED INTERSECTIONS**

| RESERVE CAPACITY (PCPH) | LEVEL OF SERVICE | EXPECTED DELAY TO MINOR STREET TRAFFIC |
|---|---|---|
| ≥ 400 | A | Little or no delay |
| 300–399 | B | Short traffic delays |
| 200–299 | C | Average traffic delays |
| 100–199 | D | Long traffic delays |
| 0– 99 | E | Very long traffic delays |
| * | F | * |

* When demand volume exceeds the capacity of the lane, extreme delays will be encountered with queuing which may cause severe congestion affecting other traffic movements in the intersection. This condition usually warrants improvement to the intersection.

Levels of service are presumed to be loosely correlated to delay. The procedure, however, does not allow for a precise quantification, and cannot be compared with the results of a signalized analysis.

The results of an unsignalized intersection analysis are somewhat less definitive than for other procedures in the manual. The HCM recommends that delay studies be included as part of the normal field data collection efforts to augment computational analysis for unsignalized intersections. This will allow more precise quantification of operational quality.

## CHAPTER 11: URBAN AND SUBURBAN ARTERIALS

Urban and suburban arterials are defined as surface highways having signal spacings of less than 2 miles. Such facilities are characterized by platoon flow. Operational quality is controlled primarily by the quality of signal coordination, and is affected by how individual signalized intersections operate along the arterial.

Arterials do not have a capacity per se. Midblock segments and characteristics do not have a measurable effect on capacity, which is controlled by the capacity of signalized intersections along the arterial. Thus, the methodology of Chapter 11 depends on individual signalized intersection analyses, and is linked to the procedures of Chapter 9. The chapter takes the results of intersection analyses and adds an evaluation of running time on arterial segments to give an overall LOS evaluation.

Figure 11-2 shows a flow chart of the procedure. Note that the procedure allows, even recommends, that field data collection be used as an alternate to the running time and intersection delay estimates computed using Chapter 9 and 11 procedures. Thus, analysis can be carried out computationally, or by the direct comparison of field data to criteria stated in the chapter.

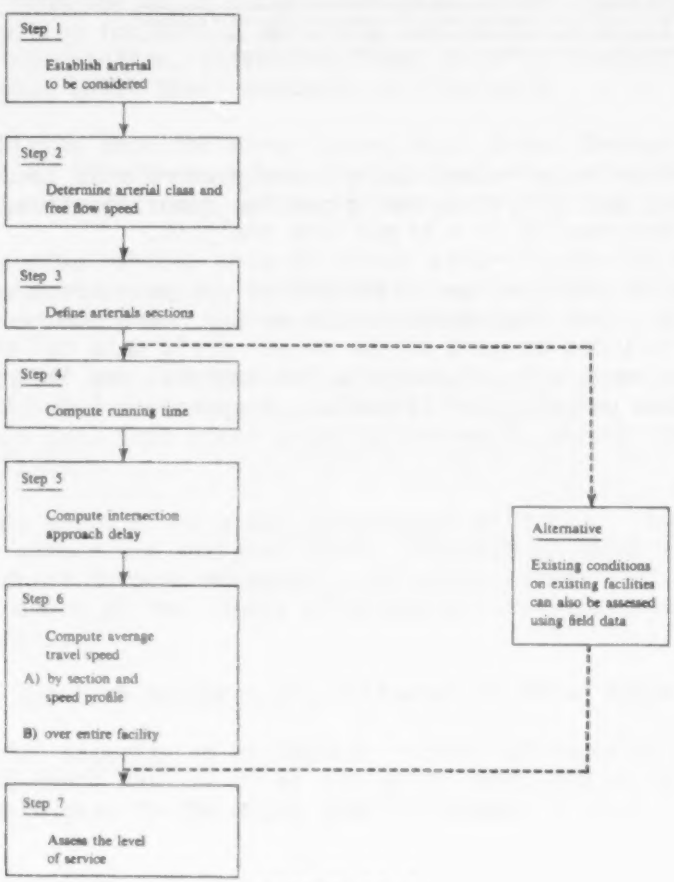



FIGURE 11-2

ARTERIAL LEVEL OF SERVICE METHODOLOGY

A significant feature of the Chapter 11 methodology is the recognition that arterials cover a wide range of facility types serving an equally wide range of functions. Service quality expectations also vary over this range of facilities. The methodology stratifies arterials into three classifications. The classifications are complex, and are related to functional and design categories. Table 11-2 describes the functional and design categories, while Table 11-3 relates these to the arterial classification.

## TABLE 11-2

## FUNCTIONAL AND DESIGN CATEGORIES FOR ARTERIALS

| CRITERIA | FUNCTIONAL CATEGORY | |
|---|---|---|
| | PRINCIPAL ARTERIALS | MINOR ARTERIALS |
| Mobility function | Very important | Important |
| Access function | Very minor | Substantial |
| Points connected | Freeways, important activity centers, major traffic generators | Principal arterials |
| Predominant trips served | Relatively long trips between above points and through trips entering, leaving, and going through the city | Trips of moderate lengths within relatively small geographical areas |

| CRITERIA | DESIGN CATEGORY | | |
|---|---|---|---|
| | SUBURBAN DESIGN | INTERMEDIATE | URBAN DESIGN |
| Control of access | Partial to almost full | Partial | Little or no control |
| Arterial type | Multilane divided; undivided or two-lane with shoulders | Multilane divided or undivided; one-way; two-lane | Undivided one-way; two-way, two or more lanes |
| Parking | No parking | Some parking | Parking permitted |
| Separate left-turn lanes | Yes | Some | No |
| Signals per mile | 1 to 4 | 4 to 8 | 8 to 12 |
| Speed limits | 40 to 45 mph | 30 to 40 mph | 25 to 35 mph |
| Pedestrian interference | None | None | Some |
| Roadside development | Low density | Moderate | High density |

## TABLE 11-3

## ARTERIAL CLASSIFICATIONS DEFINED

| DESIGN CATEGORY | FUNCTIONAL CATEGORY | |
|---|---|---|
| | PRINCIPAL ARTERIAL | MINOR ARTERIAL |
| Typical Suburban Design and Control | I | II |
| Intermediate Design | II | III |
| Typical Urban Design | III | III |

Once classified, the arterial is divided into segments for analysis. An arterial segment is a one-way section of arterial encompassing a series of blocks or links terminating at a signalized intersection. Adjacent segments of an arterial may be combined to form larger arterial sections, provided that the segments have similar demand flows and characteristics.

Level of service is related to the average travel speed of vehicles along the arterial segment or section. This is computed as follows:

$$\text{ART SPD} = \frac{3{,}600\ L}{(RT_m)\ (L) + D_A}$$

where:

ART SPD = average travel speed for the arterial segment, mph.

L = length of the arterial segment, mi.

$RT_m$ = running time per mile between signals, secs/mi.

$D_A$ = total intersection approach delay, secs.

Once the average travel speed on the arterial is determined, the LOS is found by comparing the speed to the criteria in Table 11-7. The table includes both intersection criteria and arterial criteria for convenience. Note that LOS criteria vary for the different classifications of arterial, reflecting differences in driver expectation.

## TABLE 11-7

## LEVELS OF SERVICE FOR ARTERIALS

| INTERSECTION LOS | |
|---|---|
| LEVEL OF SERVICE | STOPPED TIME DELAY PER VEHICLE (SEC) |
| A | ≤ 5.0 |
| B | 5.1 to 15.0 |
| C | 15.1 to 25.0 |
| D | 26.1 to 40.0 |
| E | 40.1 to 60.0 |
| F | > 60.0 |

| ARTERIAL LOS | | | |
|---|---|---|---|
| ARTERIAL CLASS | I | II | III |
| Range of Free Flow Speeds (mph) | 45 to 35 | 35 to 30 | 35 to 25 |
| Typical Free Flow Speed (mph) | 40 mph | 33 mph | 27 mph |
| LEVEL OF SERVICE | AVERAGE TRAVEL SPEED (MPH) | | |
| A | ≥ 35 | ≥ 30 | ≥ 25 |
| B | ≥ 28 | ≥ 24 | ≥ 19 |
| C | ≥ 22 | ≥ 18 | ≥ 13 |
| D | ≥ 17 | ≥ 14 | ≥ 9 |
| E | ≥ 13 | ≥ 10 | ≥ 7 |
| F | < 13 | < 10 | < 7 |

It should be noted that intersection and arterial LOS do not always correlate. For example, an arterial with many signalized intersections may operate at a considerably worse LOS than the individual intersections. This is a reflection of the deleterious impact of many closely-spaced signals on arterial flow.

As with other procedures in the 1985 HCM, worksheets are provided to assist the analyst in making and summarizing computations.

## CHAPTER 12: TRANSIT CAPACITY

The 1965 HCM had a brief chapter covering some basic concepts of bus transit capacity. The 1985 HCM has greatly expanded this material. Chapter 12 constitutes a "mini-manual" on transit capacity, covering a wide range of concepts and topics relating to bus, light rail, and rail capacity. The primary focus of the chapter is on bus transit. Some of the subjects treated in the chapter are:

1. The concept of person-capacity.
2. The definition of transit levels of service.
3. Transit vehicle capacities.
4. Bus boarding and unloading times.
5. Bus berth and system capacity.
6. Discussion of and criteria for bus priority treatments.
7. Impacts of buses on traffic.
8. Overview of rail and light rail capacities.
9. Transit characteristics and operating experience.

The concept of transit capacity is illustrated in Figure 12-2. The concept is two-dimensional, dealing with both the internal environment of the bus and the traffic environment of the bus itself. During peak periods of transit usage, the level of service usually lies in the range where buses are both crowded and subject to roadway congestion. General criteria for local buses on city streets in summarized in Table 12-12.

The "Transit" chapter covers a broad range of material on transit capacity and operations, with over 40 tables and illustrations, numerous sample calculations, 24 analytic relationships, and over 40 references for users interested in greater background and depth. The illustrations included here are examples of the thoroughness with which the subject is treated in the 1985 HCM.

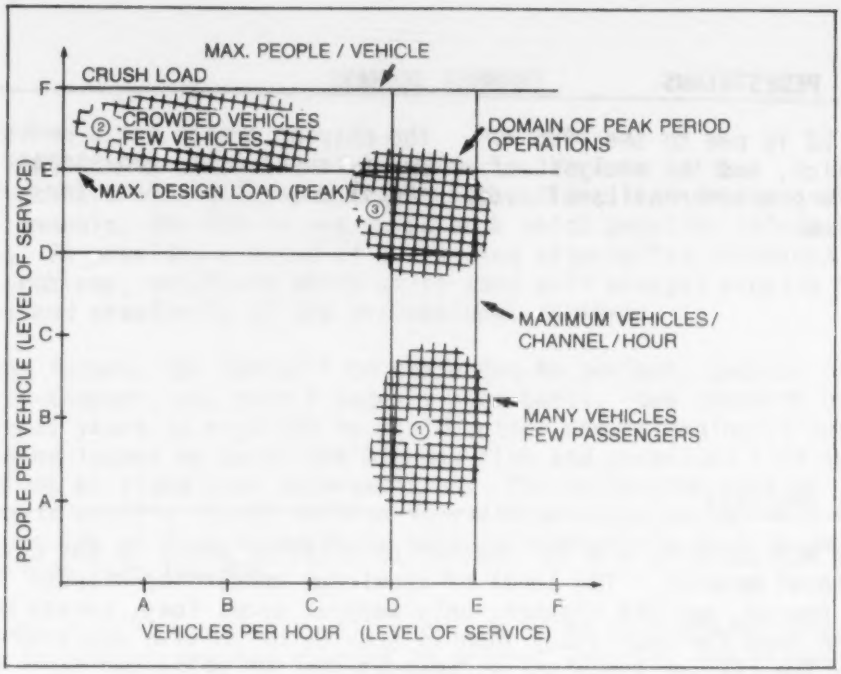



FIGURE 12-2

THE CONCEPT OF TRANSIT LEVEL OF SERVICE

TABLE 12-12

LEVELS OF SERVICE FOR LOCAL TRANSIT BUSES

| LEVEL OF SERVICE (STREET) | BUSES/ PASSENGERS/SEAT | LEVEL OF SERVICE (PASSENGERS) | | | | |
|---|---|---|---|---|---|---|
| | | A 0.00–0.50 | B 0.51–0.75 | C 0.76–1.00 | D 1.01–1.25 | E 1.26–1.50 |
| ARTERIAL STREETS | | | | | | |
| A | 25 or less | 625 | 940 | 1,250 | 1,560 | 1,875 |
| B | 26 to 45 | 1,125 | 1,690 | 2,250 | 2,810 | 3,375 |
| C | 46 to 80 | 2,000 | 3,000 | 4,000 | 5,000 | 6,000 |
| D | 81 to 105 | 2,625 | 3,940 | 5,250 | 6,560 | 7,875 |
| E | 106 to 135 | 3,375 | 5,060 | 6,750 | 8,440 | 10,125 |
| CBD STREETS | | | | | | |
| A | 20 or less | 500 | 750 | 1,000 | 1,250 | 1,500 |
| B | 21 to 40 | 1,000 | 1,500 | 2,000 | 2,500 | 3,000 |
| C | 41 to 60 | 1,500 | 2,250 | 3,000 | 3,750 | 4,500 |
| D | 61 to 80 | 2,000 | 3,000 | 4,000 | 5,000 | 6,000 |
| E | 81 to 100 | 2,500 | 3,750 | 5,000 | 6,250 | 7,500 |

NOTE: Ratio shown for level of service (passengers) is "passengers per seat" on average bus. Thus 1.00 means 50 passengers for the assumed 50 seats.
Values would be 6 percent higher for a 53-seat bus.
Values for articulated buses would be 15 to 20 percent greater.

## CHAPTER 13: PEDESTRIANS

Chapter 13 is new to the 1985 HCM. The chapter treats basic pedestrian characteristics, and the analysis of walkways, crosswalks, and street corners. The characteristics of pedestrian flow parallel those for vehicular flow.

## CHAPTER 14: BICYCLES

The 1985 HCM contains a brief chapter on bicycles which is not covered in instructional modules. The level of knowledge concerning bicycle capacity is sparse, and the chapter, only several pages long, serves more to highlight the need for more study than to provide definitive analysis procedures. The chapter contains recommendations for passenger car equivalent values for bicycles, which may be incorporated into analysis of various types of highway facilities. General guidelines for bikeway capacity are also included.

## GENERAL SUMMARY

The 1985 HCM is a major evolutionary step forward in the state-of-the-art of highway and traffic operational and design analysis. Like all manuals, the HCM is not a cookbook which provides solutions to problems. It provides a means of evaluating alternative solutions to traffic problems, solutions which still (and will always) require the expertise and creativity of the professional engineer.

In the future, the HCM will be subjected to periodic updates on a chapter-by-chapter, and even a page-by-page basis. New research over the next several years is expected to yield better understanding of such wide-ranging issues as multilane highway flow and permitted left-turn interactions at signalized intersections. The profession will no longer have to wait until a fourth edition is ready as a whole, but will be able to use the results of this research as soon as appropriate text is available for insertion into the HCM.

Highway capacity analysis will continue to be an evolving process, changing to reflect ongoing changes in driver, vehicle, and highway characteristics. It will continue to be a somewhat imprecise science, as it must address the uncertainties of complex human behavior in a complicated environment. With the 1985 HCM, engineers are able to perform some analyses never before possible, and all analyses better than previously.



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